Appendix E
Tidal and Flood Hydraulic Modeling

Introduction

This appendix presents an overview of the development and application of the North Delta tidal and flood hydraulic model. The model, built on MIKE 11 modeling engine platform, was used for evaluation of tidal and flood hydraulic impacts from the North Delta Flood Control and Ecosystem Restoration Project Alternatives. The following information is provided in this appendix; the theoretical basis of the MIKE 11 model engine, development of the North Delta Project area MIKE 11 hydraulic model, calibration and validation of the model, model inputs and assumptions, and flood control and ecosystem restoration modeling results. Most of the work described herein was completed throughout the course of three University of California at Davis (UCD) Masters theses. Sediment transport and water quality modules of the MIKE 11 have also been developed to analyze changes/impacts in sediment transport and sediment budget for different proposed Project Alternatives. The sedimentation study has been discussed in Chapter 3 of the EIR.

MIKE 11 Model

The MIKE 11 model (DHI 2000), developed by the Danish Hydraulic Institute, is a dynamic, one-dimensional modeling package, which simulates the water level and flow splits throughout a river/channel system. In addition to simulating hydraulics, the modeling package also includes modules for advection-dispersion, sediment transport, water quality, rainfall-runoff, flood forecasting, and GIS floodplain mapping and analysis. The hydraulic and sediment transport modules were developed and used to analyze potential impacts and benefits of the North Delta Project.

MIKE 11 solves the vertically integrated equations of conservation of mass and momentum, known as the St. Venant equations. The St. Venant equations are derived from the standard forms of the equations of conservation of mass and conservation of momentum based on the following four assumptions:

- The water is incompressible and homogeneous; therefore, there is negligible variation in density.

- The bottom (channel bed) slope is small, therefore the cosine of the slope angle can be assumed to equal 1.

- The water surface wavelengths are large compared to the water depth, which ensures that the flow everywhere can be assumed to move in a direction parallel to the bottom.

- The flow is subcritical. Subcritical flow conditions are solved with a reduced momentum equation, which neglects the nonlinear terms.

With the four assumptions applied, the standard forms of the equations of conservation of mass and momentum can be transformed into the equations below. These transformations are made with Manning's formulation of hydraulic resistance in SI units, and the incorporation of lateral inflows in the continuity equation.

Continuity Equation: $\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q$

Momentum Equation: $\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} + \frac{n^2 gQ|Q|}{AR^{4/3}} = 0$

where

Q: discharge [ft^3/s]	α: vertical velocity distribution coefficient
A: cross section area [ft^2]	g: gravitational acceleration [ft/s^2]
X: downstream direction [ft]	h: stage above datum [ft]
t: time [s]	n: Manning coefficient
q: lateral inflow [ft^2/s]	R: hydraulic radius [ft]

Within the MIKE 11 program, the above equations are transformed into a set of implicit finite difference equations, which are solved for each point in the grid (at each node). The above formulations of the St. Venant equations are further simplified for application in a rectangular channel. Natural river cross sections are rarely rectangular, so the MIKE 11 model integrates the equations piecewise in the lateral direction. In order to run the MIKE 11 model, several data inputs are required, including the river network alignment, channel and floodplain cross sections, boundary conditions and roughness coefficients.

The MIKE 11 GIS software package integrates MIKE 11 hydraulic model output with the spatial analysis capabilities of the Arc View GIS software developed by

Environmental Science Resource Institute. MIKE 11 GIS, among other things, projects the water levels calculated within MIKE 11 as an interpolated water surface over a digital elevation model (DEM). The difference between the water level and the ground elevation is determined throughout the domain and visually presented based upon user defined flood depth increments. This software is designed to assess flood extent and provide insight with regards to the regional ecology driven by the disturbance of flooding. For example, depth inundation maps have been generated with MIKE 11 GIS to evaluate the habitat restoration potential of North Delta ecosystem restoration scenarios on McCormack-Williamson Tract. This provides a powerful graphical tool when evaluating each scenario based upon defined management objectives.

North Delta MIKE 11 Model Development

UCD staff worked cooperatively with DWR staff and the Project area stakeholders to develop the MIKE11 model. Model development was completed through the grant-funded work of several graduate students whose efforts built upon the others in succession. The students' work is documented in three Masters theses: "An Unsteady Hydraulic Surface Water Model of the Lower Cosumnes River, California, for the investigation of floodplain dynamics," by Stephen H. Blake; "Hydrodynamic Modeling and GIS Analysis for the Habitat Potential and Flood Control Benefits of the Restoration of a Leveed Delta Island," by Chris T. Hammersmark; and "Water Quality Modeling and Monitoring in the California North Delta Area," by Raffi J. Moughamian.

The North Delta MIKE11 modeling efforts described in this Appendix were coordinated with other area modeling efforts, such as the development of a regional HEC-RAS, a one-dimensional hydraulic model developed by US Army Corps of Engineers. Most of the channel geometry and boundary condition for the North Delta MIKE11 model were obtained from those kinds of efforts.

Project Area

The Project area lies within Sacramento and San Joaquin Counties. The Cosumnes River, its forks, and tributaries extend into the counties of El Dorado and Amador, with the uppermost reaches of the Mokelumne found in Calaveras and Alpine counties (Blake 2001). Project area watersheds, including Cosumnes and Mokelumne River watersheds, are shown in Figure E-1.

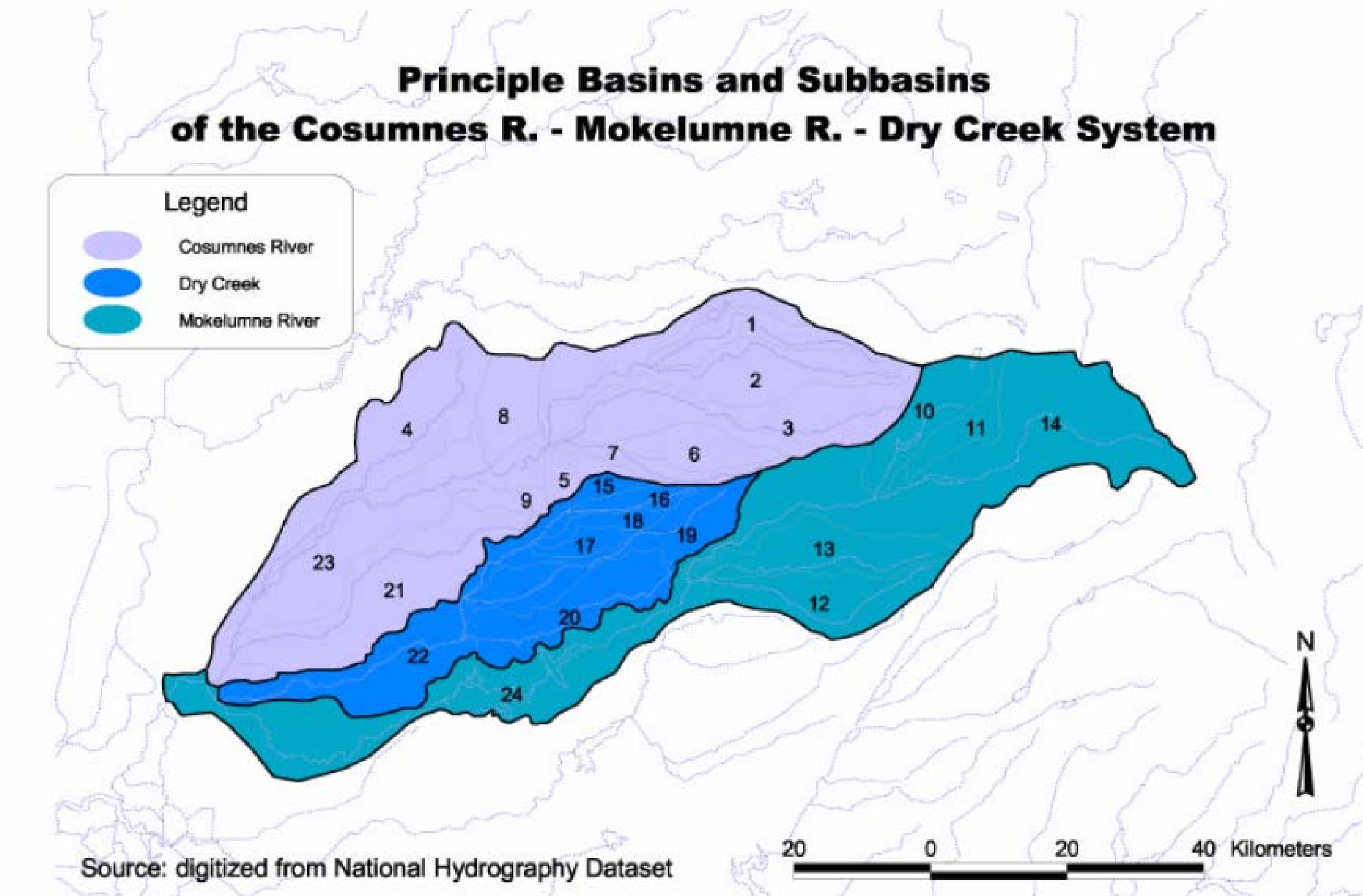


Figure E-1 Principle Basins and Subbasins of the Project Area

Model Geometry

The alignment of river channels, major sloughs, and floodplain areas in the North Delta model region dictates the model network of the hydraulic system for the Project (shown in Figure E-2). A total of 150 miles of river channels and sloughs are included in the model, not including the extensive off channel regions, which are also incorporated in the model network. The model utilizes 454 in-channel and floodplain cross sections obtained from a variety of sources (Hammersmark 2002). All cross section and boundary data are datum verified and translated as needed to the NGVD 29 datum (mean sea level).

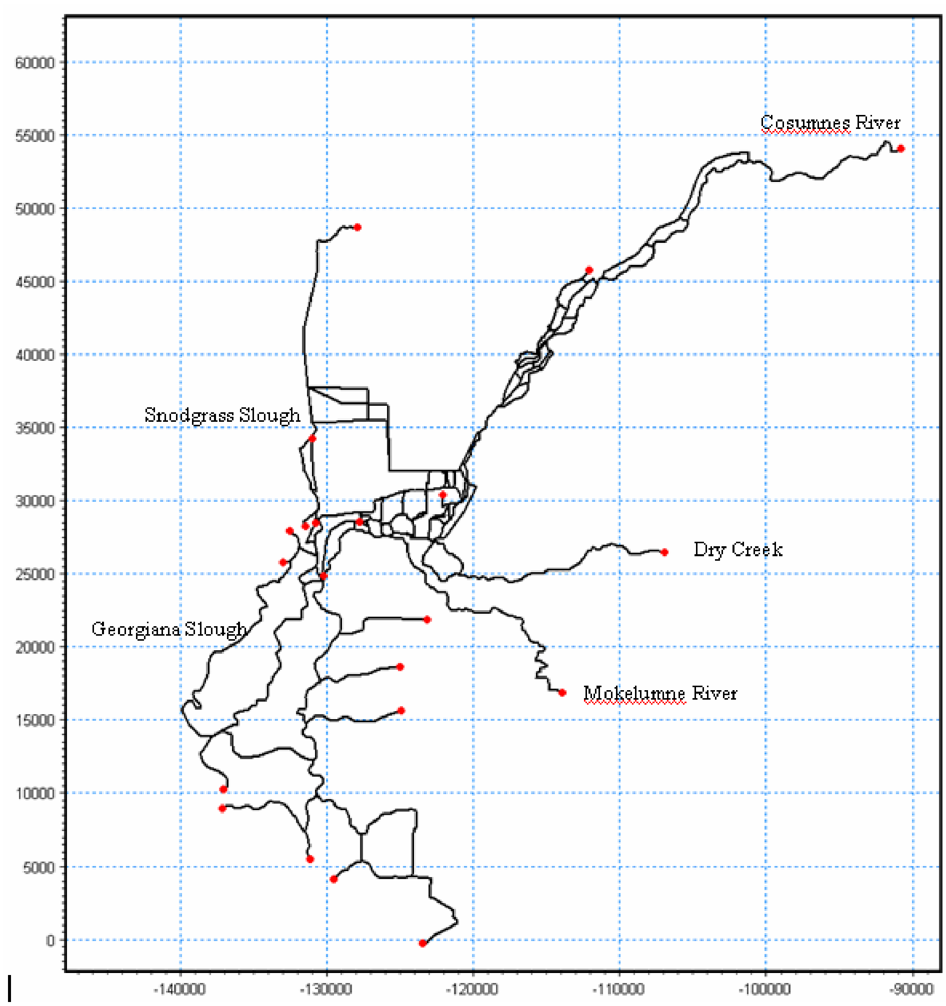


Figure E-2: North Delta MIKE11 Model Schematic (Model Domain)

Each river reach/branch is assigned a name and length in addition to its connectivity with the other branches in the model domain. The model incorporates the Cosumnes and Mokelumne Rivers, Dry Creek, Georgiana Slough, Snodgrass Slough, Morrison Creek Stream Group, the San Joaquin River, and many backwater sloughs to capture the hydrodynamics in the North Delta area. In this study, floodplains are identified as separate reaches in the model network, placed adjacent to the channel. The floodplain is then connected to the river reach with “link channels”, which are basically simplified branches in which flow through the branch is calculated as flow over a broad crested weir, with user defined weir geometry. All levee breaches, in addition to floodplain connections have been simulated with this approach, providing a pseudo two-dimensional representation of floodplain flow. Detailed information on the model branch names, chainages, flow directions, and network connectivity can be found in Hammersmark (2002).

Topographic and cross section data for the original model development are detailed in Appendix A of the Stephen Blake thesis. Geometric data in the form of cross sections and digital elevation models from a variety of sources including USGS, CA-DWR, University of California at Davis (UCD), EBMUD, SAFCA, Phillip Williams and Associates (PWA), California Department of Transportation BIRIS system (BIRIS), Sacramento County Public Works Department, San Joaquin County Public Works Department, and the National Oceanic and Atmospheric Administration (NOAA) were used to develop the model. The data was collected in various forms such as DEMs, AutoCAD drawings, binary data sets used in other modeling platforms, field surveys, as-built drawings of bridges, and output from an NOAA NOS lidar mission. The data were location and datum verified, processed, and compiled into a cross-sectional database in MIKE 11. Figure E-3 presents the location and source (where available) of each cross section used in this effort.

Topographic data for large floodplain areas where no formal survey data exists were extracted from the USGS 30-meter DEM. These areas include Glanville Tract, Dead Horse Island, Erhardt Club, New Hope Tract, and Tyler Island. Topography data for the McCormack-Williamson Tract were obtained from the North Delta Study conducted in 1992 by DWR, and then partially verified for significant changes in the topography from the original survey (Hammersmark 2002).

Boundary condition data were gathered from a number of gages in the North Delta Project area. Those data were provided by a number of agencies including United States Geological Survey (USGS), California Department of Water Resources (CA-DWR), East Bay Municipal Utilities District (EBMUD), and Sacramento County Flood Control Agency (SAFCA). The availability of hydraulic gage data somewhat dictates the boundaries of the North Delta MIKE 11 model domain. The model extends upstream to hydraulic gages located at Michigan Bar on the Cosumnes River, Wilton Road on Deer Creek, above Galt on Dry Creek, Woodbridge on the Mokelumne River, and to Lambert Road at the Stone Lakes Outfall. To the west, the model includes a short portion of the Sacramento River extending from above the Delta Cross Channel to below the divergence of Georgiana Slough. There are four downstream boundary conditions on the San Joaquin River including the San Joaquin River at San Andres Landing, Venice Island, Turner Cut, and Rindge Pump. Gage data from two internal locations, Benson’s Ferry and New Hope, were used as calibration and verification points. Figure E-4 shows the locations of the North Delta MIKE11 boundary conditions. Types of boundary condition data used are listed in Table E-1.

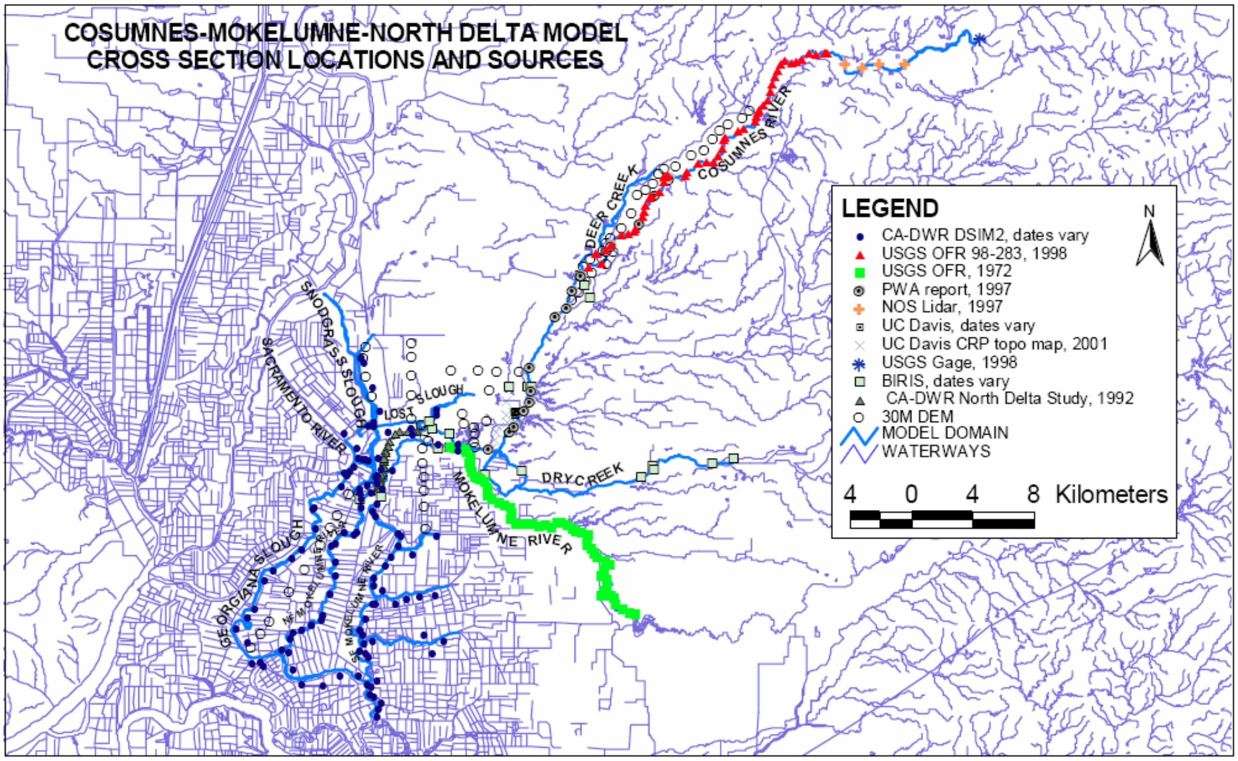


Figure E-3 Cross section locations and data sources used in the North Delta Model.

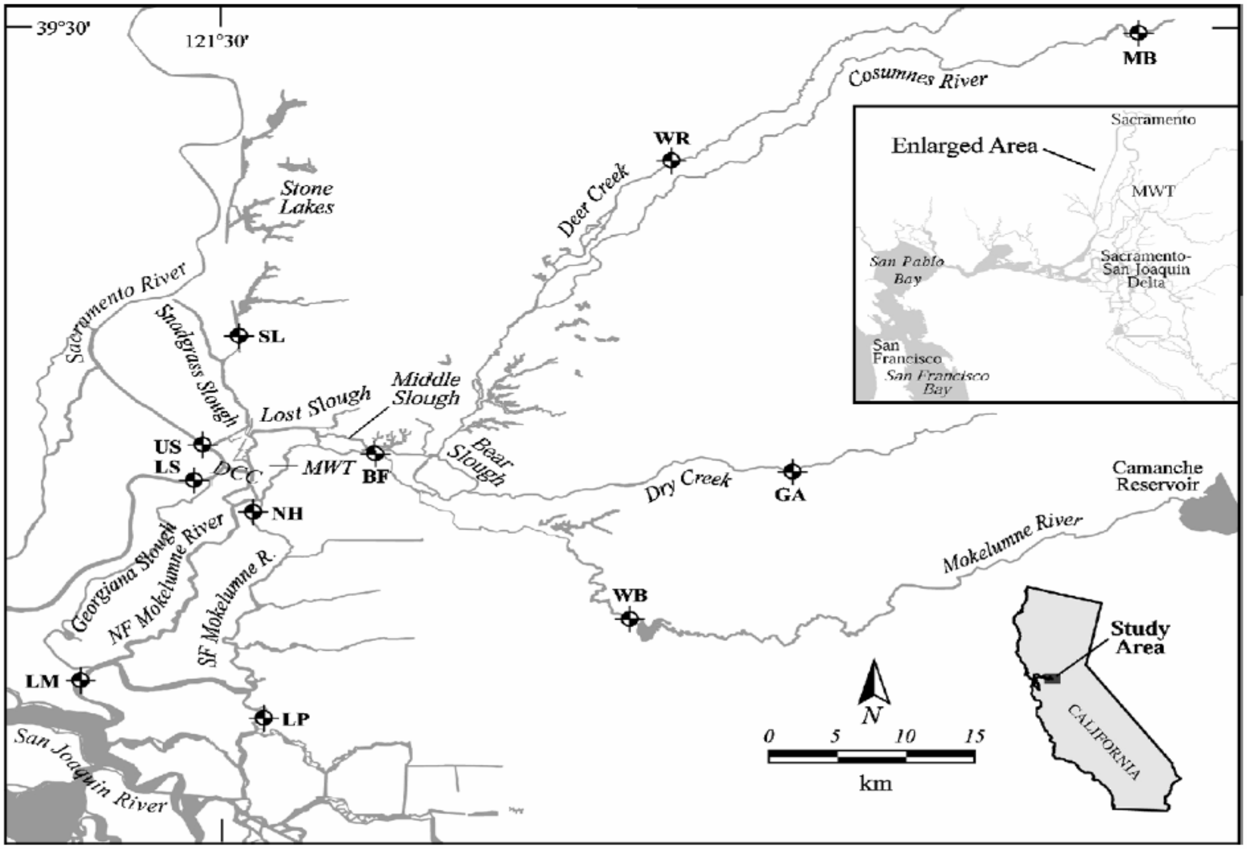


Figure E-4 Regional and local setting of the McCormack-Williamson Tract and location of gages used for boundary conditions and internal validation points. Model result validation and scenario comparison is conducted at Benson's Ferry (BF) where the Cosumnes River converges with the Mokelumne River and at New Hope (NH) where the North and South Forks of the Mokelumne River diverge. Model boundary conditions are labeled as follows: MB: Michigan Bar on the Cosumnes River, WR:Wilton Road on Deer Creek, GA: Galt on Dry Creek, WB:Woodbridge on the Mokelumne River, SL: Stone Lakes Outlet at Lambert Road, US: Sacramento River above the Delta Cross Channel (DCC), LS: Sacramento River below Georgiana Slough, LM: Lower Mokelumne River at Georgiana Slough and LP: Little Potato Slough below Terminous.

Table E-1: Hydraulic Model Boundary Condition Data Type

Hydraulic Gage Location	Sensor ID	Agency	Simulation Year/Data Type				
			1986	1997	1998	1999	2000
Upstream Boundary							
Cosumnes River @ Michigan Bar	RCSM075	USGS	Q&h	Q&h	Q&h	Q&h	Q&h
Sacramento River upstream of the DCC	RSAC128	USGS	--2	Q&h	Q&h	Q&h	Q&h
Dry Creek upstream of Galt	DRY1	USGS	Q	e	e	e	e
Mokelumne River at Woodbridge	RMKL070	EBMUD	Q&h	Q&h	Q&h	Q&h	Q&h
Deer Creek at Wilton Road	DEER2	SAFCA	E	Q&h	Q&h	Q&h	Q&h
Stone Lakes Outlet at Lambert Road	SGS1	SAFCA	e	h	h	H	h
Downstream Boundary							
Sacramento River downstream of Georgiana Slough	RSAC121	USGS	h	Q&h	Q&h	Q&h	Q&h
San Joaquin River at San Andres Landing	B95100	DWR	h	h	h	h	h
San Joaquin River at Venice Island	B95580	DWR	h	h	h	h	h
San Joaquin River at Turner Cut	--	DWR	h	h	h	h	h
San Joaquin River at Rindge Pump	B95620	DWR	h	h	h	h	h
Internal Boundary							
Mokelumne River at Benson's Ferry	RMKL027	DWR	h	h	h	h	h
South Fork Mokelumne River at New Hope Landing	RSMKL024	DWR	h	h	h	h	h

1) Q = discharge, h = stage, e = estimated as explained in text

2) For the 1986 simulation, stage data at Sacramento River downstream of Georgiana Slough were used for the upstream end of Georgiana Slough and the Sacramento River reach was removed from the model network.

Data collected at different times, and by different agencies does not always utilize the same reference datum, and in some cases does not document the reference datum used. To ensure uniformity and confidence in the modeling results, data from each source have been datum checked and converted as needed to the National Geodetic Vertical Datum of 1929 (NGVD 29).

Bridges and Structures

All bridges and structures were included in the model as cross-sections to allow the model to calculate the effects of the restrictions. The data for the bridges came from the State and County drawings available for the structures, and the data for the DCC from the USBR ‘as built’ drawing number 214-D-16819.

Roughness Coefficients

The MIKE11 model requires the input of channel roughness in each reach for calculating water surface elevations. Roughness values were input by designating a roughness coefficient, Manning’s n for each reach. The value of this coefficient depends on many things, but primarily upon bed and bank materials, the amount of vegetation, and channel irregularity. For this Project, a number of n-value tables and photographs were used to estimate “n” values for various regions of the model domain. The final values are shown in Table E-2. More detail on the method of choosing the Manning’s n values is given in Hammersmark (2002).

Table E-2 North Delta MIKE 11 Manning Coefficients

	Global value[1]	Cosumnes River[2]	Deer Creek	Dry Creek	Delta Islands and Tracts	Floodplains
Manning’s n	0.036	0.04	0.05	0.05	0.05	0.1

[1] The global value was applied to all model regions unless otherwise specified.

[2] For the 1986 runs, Cosumnes River “n” value was increased to 0.045 to account for the increases effect of vegetation at high water levels.

Calibration and Validation of the Model

For a successful comparative evaluation of Project Alternatives, it is important to have a well calibrated and validated hydraulic model. The MIKE 11 model for the North Delta Project was calibrated and validated for a range of flows to ensure that the model was capable of simulating a range of storm events. This section documents the flow data used for calibration and validation, the methodology, and comparisons between model outputs and the measured data.

Flow Data

The range of flows, considered for modeling the Project Alternatives, varies from a 2.5-year to over 200-year return interval at Michigan Bar. The return interval for various flood pulses at Michigan Bar has been chosen as the distinguishing variable because the Cosumnes River is the dominant source of floodwater to the North Delta region. Michigan Bar has a comparatively long record of gage data. The return interval or flood recurrence interval is defined as the expected period of time within which a flood of a given magnitude will be equaled or exceeded. In other words, the chance that a 50-year recurrence interval flood will occur in a given year is 1 in 50.

Flood frequency analyses were performed by the USGS for the Cosumnes River based upon 91 years of data (1907-1997) recorded at the Michigan Bar gaging station (Guay et al. 1998). Philip Williams and Associates (PWA) performed another flood frequency analysis for the Cosumnes River based upon 89 years of data (1907-1995) recorded at the Michigan Bar gaging station (Vick et al 1997). As well, David Ford Consulting Engineers Inc. performed a flood frequency analysis as part of work prepared for Sacramento County. These flow frequency analyses have been used to describe the recurrence intervals of flood pulses in this study. Of note, all the analyses clearly show that the peak Michigan Bar flow for 1997, which was reported at 93,000 cfs, significantly exceeded a 100 year event and the two most recent analyses (PWA and David Ford) have the 1997 event exceeding a 200 year event. Table E-3 shows the peak flows for different return intervals for Michagan Bar from the various analyses.

Table E-3 Comparison of peak flow (cfs) at Michigan Bar

	Return Period (Year)					
	10	**25**	**50**	**100**	**200**	**500**
USGS	34,200		66,800	82,900		125,000
PWA	30,548			68,000	79,900	
David Ford	40,846	53,865	60,400	73,022	82,340	

Index Points

In addition to utilizing gage data as boundary conditions for the simulated hydraulic system, gage data from locations within the model domain, including Benson's Ferry and New Hope Landing, were used to calibrate and validate the model results. Figure E- 5 shows the index points that were used in the model to interpret and compare results for different Project Alternatives.

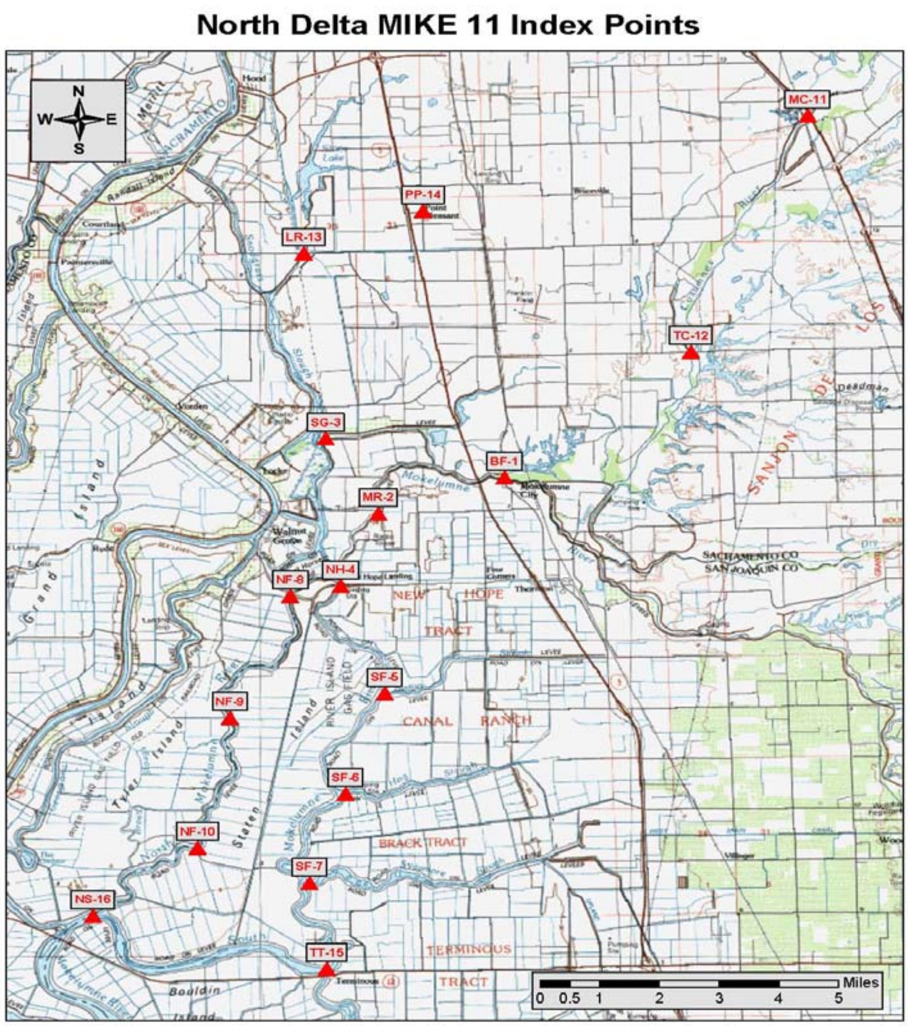


Figure E-5 North Delta MIKE11 Index Points

Model Limitations

One-Dimensional Model

It is also important to understand the simplifications and assumptions which are often made when applying a model and evaluating a physical system. The MIKE 11 hydraulic model used for the North Delta Project area is hydraulic not hydrologic. Hydrologic elements of river and floodplain systems, which are not incorporated, include the groundwater-surface water interaction, as well as surface water interaction with the atmosphere and vegetation. Water movement is simulated based upon water forces, and assumed to act only in the longitudinal direction. Thus effects from an eddy or a rapid, formed by a constriction in the river channel or at a levee breach are not captured in this model (or in any one dimensional hydrodynamic model).

Cross Sections and Boundary Conditions

A great deal of real data have been utilized in compiling, calibrating, and validating the model. However, many crucial data elements including cross sectional geometry, boundary conditions, and system connectivity are not available, and hence, have been estimated. Other uncertainties arise when using cross sectional data, which were measured at different times with different methods. For example, data from as early as 1934 were used in the model. Yet another element of uncertainty is the lack of channel cross sectional data in some reaches, with 2.1 miles between cross sections in some cases.

Estimation of certain boundary condition data was necessary. Boundary condition estimation was required for Deer Creek at Wilton Road, Dry Creek above Galt, Stone Lakes Outfall at Lambert Road, and Little Potato Slough below Terminous Tract, for various time periods of the 1986, 1997, 1998, 1999, and 2000 storm events.

Dry Creek Flow

The Dry Creek watershed is known to contribute significant flows to the North Delta Project area during storms. Gage data at the Dry Creek Galt gage is available for limited periods. Data for the gage during the 1986 storm is available, but in order to simulate the years of 1997, 1998, 1999, and 2000 an estimation of the Dry Creek flow contribution was required. A comparison of daily average discharge values in 1986 suggests that during storm events, the Dry Creek at Galt discharge is roughly 40% of Cosumnes River discharge at Michigan Bar. Based upon this comparison of historic discharge data the Dry Creek at Galt boundary condition were estimated for the 1998, 1999, and 2000 model runs to be 40% of the discharge of the Cosumnes River at Michigan Bar

(USACE 1990). However, 30% of the Michigan Bar discharge was used for the 1997 run. A limitation to this approach is that it overestimates Dry Creek discharge during low flow conditions, and may underestimate Dry Creek discharge during flood pulses.

Stage Data

Data from the stage gages located at Wilton Road on Deer Creek and Lambert Road at the Stone Lakes Outfall, both operated by SAFCA, do not exist for 1986. For the Wilton Road gage, a correlation to an adjacent gaging station for which data were available was not attempted. Instead, an average low flow water elevation of 53.8 feet was assumed. This value was chosen by inspection of available data for the period of 1998-2000. No attempt was made to synthesize flood pulse water levels. At the Stone Lakes Outfall at Lambert Road, a control structure prevents water from flowing south to north at this location. For a brief period during the large flood of 1986, flow traveled over Lambert Road north into the Stone Lakes Region (USACE 1988). For 1986 model simulations a weir was inserted at Lambert Road, which prevented flow during non-flood conditions, but allowed some water to travel north over Lambert Road during the peak of the flood pulse (Hammersmark 2002).

Calibration Methodology

The high degree of uncertainty in various model inputs such as channel geometry, assumed boundary conditions, and system connectivity, made calibration and verification of the model a complex undertaking. The model improvement and calibration proceeded in two phases, focusing on different flow conditions. Initially, the low flow, tidally dominated portion of the hydrograph was considered, and adjustments were made so that the model would accurately reflect the amplitude and timing of observed tidal signal data.

The second phase of model calibration focused on improving the timing, magnitude and hydrograph shape of various flood pulses. This involved refining the connectivity of the simulated hydraulic system to result in the best agreement with observed data. In particular, the manner in which the Cosumnes River channel flow accesses (through overtopping, breaching, etc.) floodplain regions, and the effect of such regions on attenuating flood pulses was refined. (Hammersmark 2002)

Comparison to Observed Data

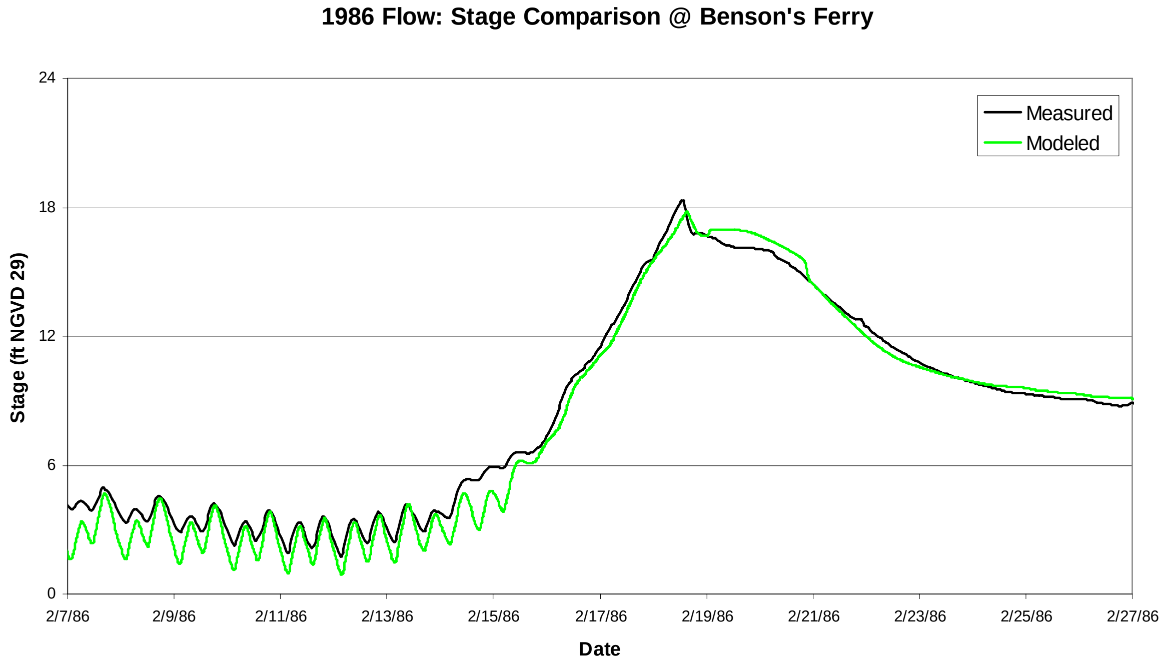
Ultimately, the North Delta MIKE 11 model was applied to simulate the flooding period of the following five years: 1986, 1997, 1998, 1999, and 2000. Calibration plots (shown in Figures E-6 through E-10) illustrate that the model is in good agreement with the observed data for the range of storm events. They include tidal influence and floods of various magnitudes, including two large storm events (1986 and 1997). Deviations in some of the peaks are most likely the result of the use of a constant percentage of Michigan Bar flows applied for Dry Creek. There was no apparent basis to manipulate the Dry Creek flows for year to year to better represent the flow ranges. The observed agreement of the model results with the measured data ensured that it could be confidently used for the comparative evaluation of flood control and ecosystem restoration Alternatives.

One additional method of evaluating the model results for the 1986 flooding event was a comparison of maximum floodwater volume stored in the various areas flooded as levees failed. Maximum floodwater storage in McCormack-Williamson Tract, Glanville Tract, Dead Horse Island, Tyler Island, and New Hope Tract were estimated by the Sacramento District of the U. S. Army Corp of Engineers (1988). Table E-4 presents the values that support a reasonable agreement between the estimate and the model.

Table E-4 Comparison of model simulation results to estimated values of maximum floodwater storage for each flooded island or tract during the 1986 flood event.

Flooded Region	Maximum Floodwater Storage (ac-ft)	
	Simulation	Estimated [1]
Glanville Tract	48,900	45,000
M-W Tract	18,900	17,000 – 20,000
Dead Horse Island	2,700	2,000 – 3,000
Tyler Island	108,000	130,000 -150,000
New Hope Tract	49,300	60,000

Note:
[1] Estimated maximum floodwater storage values obtained from U. S. Army Corps of Engineers, 1988.



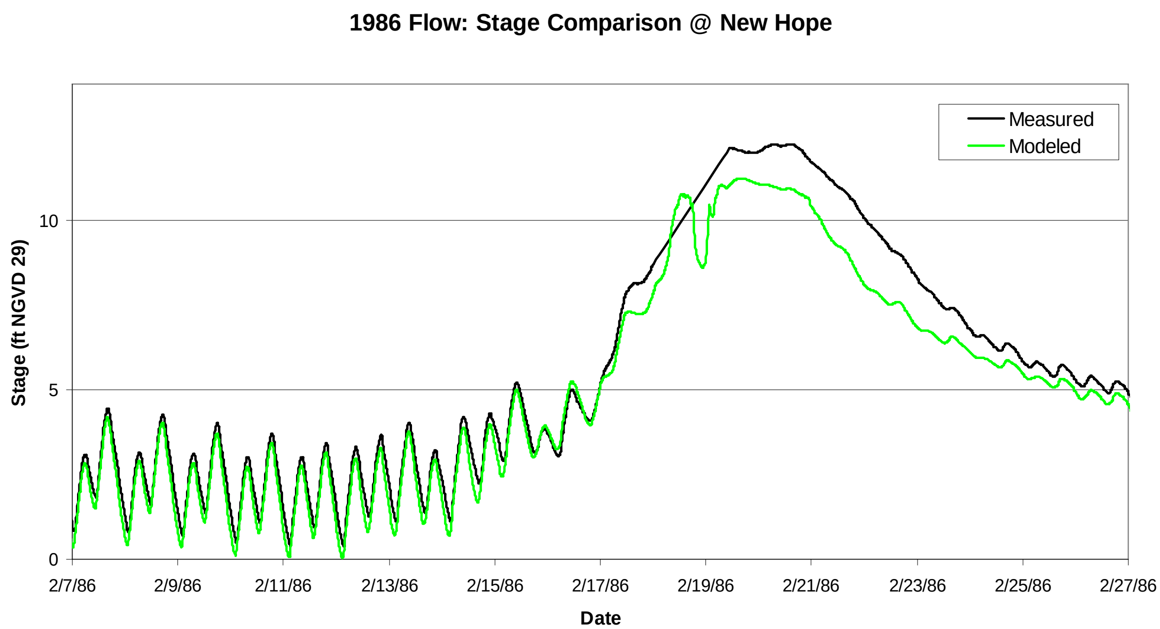
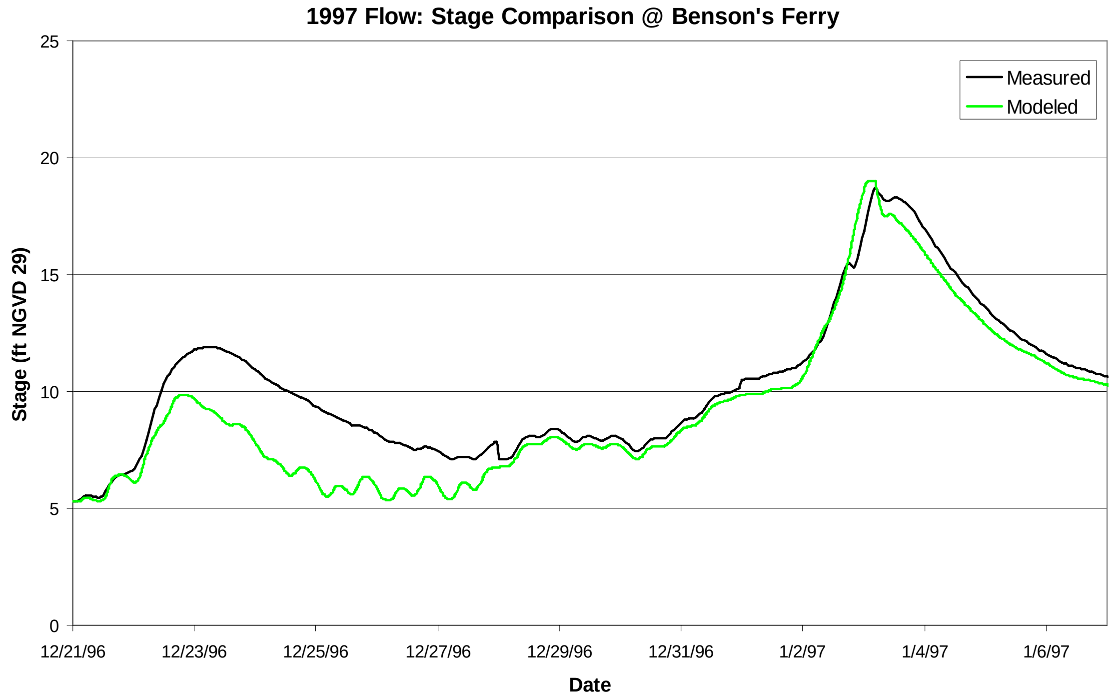


Figure E-6 Model results compared to measured data at Benson's Ferry (top panel) and New Hope (bottom panel) for the year 1986 flow.



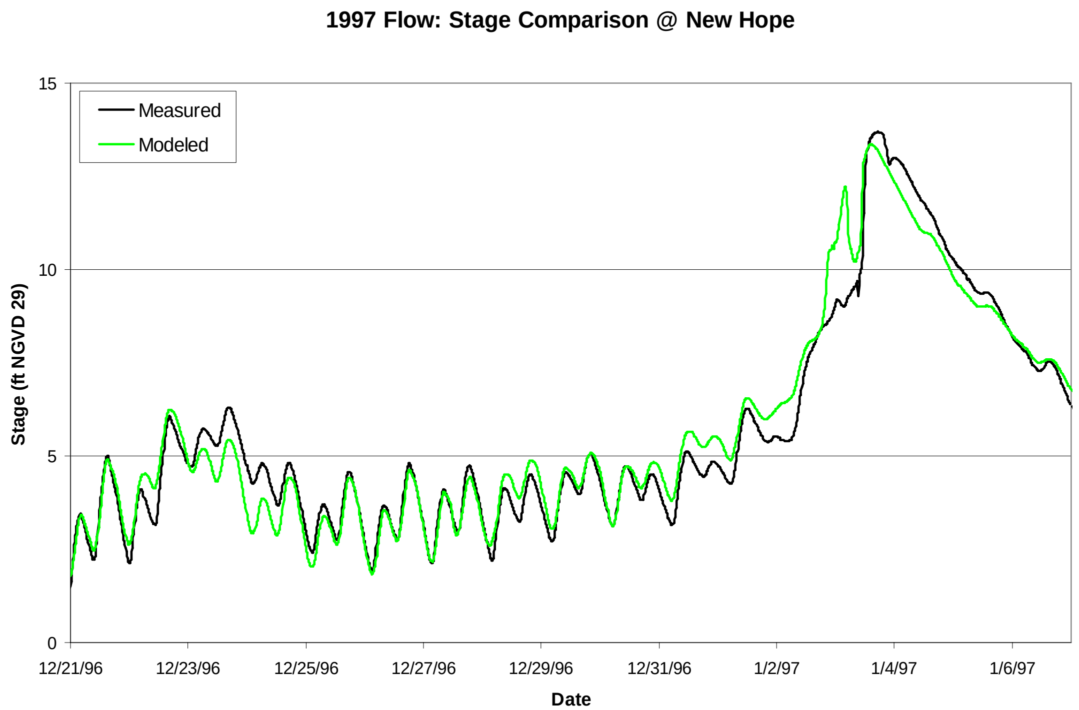
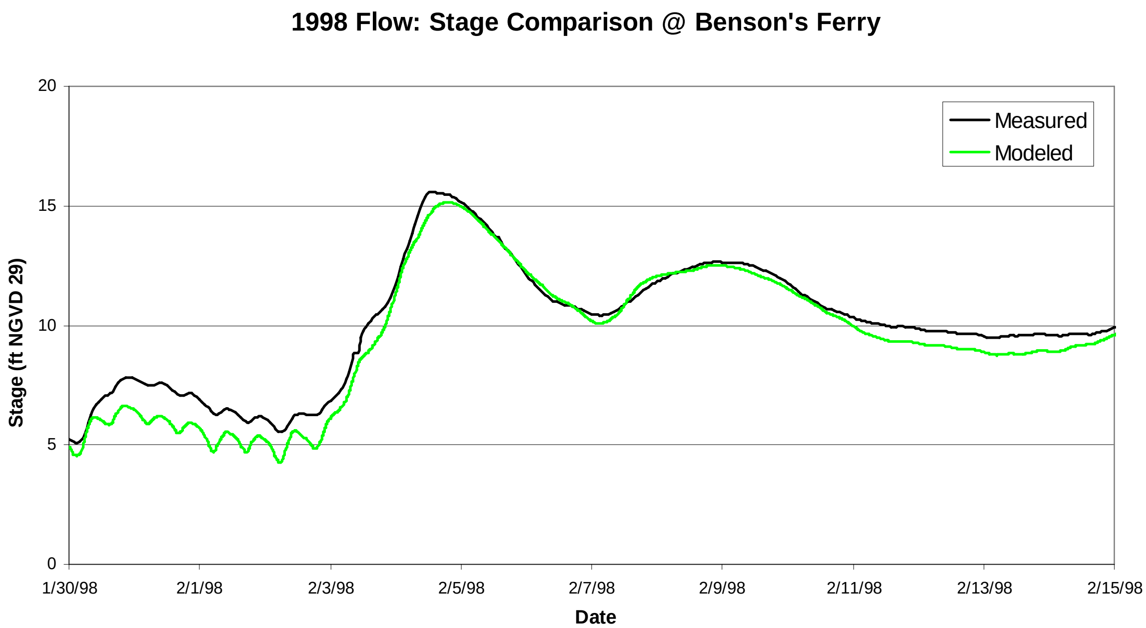


Figure E-7 Model results compared to measured data at Benson's Ferry (top panel) and New Hope (bottom panel) for the year 1997 flow.



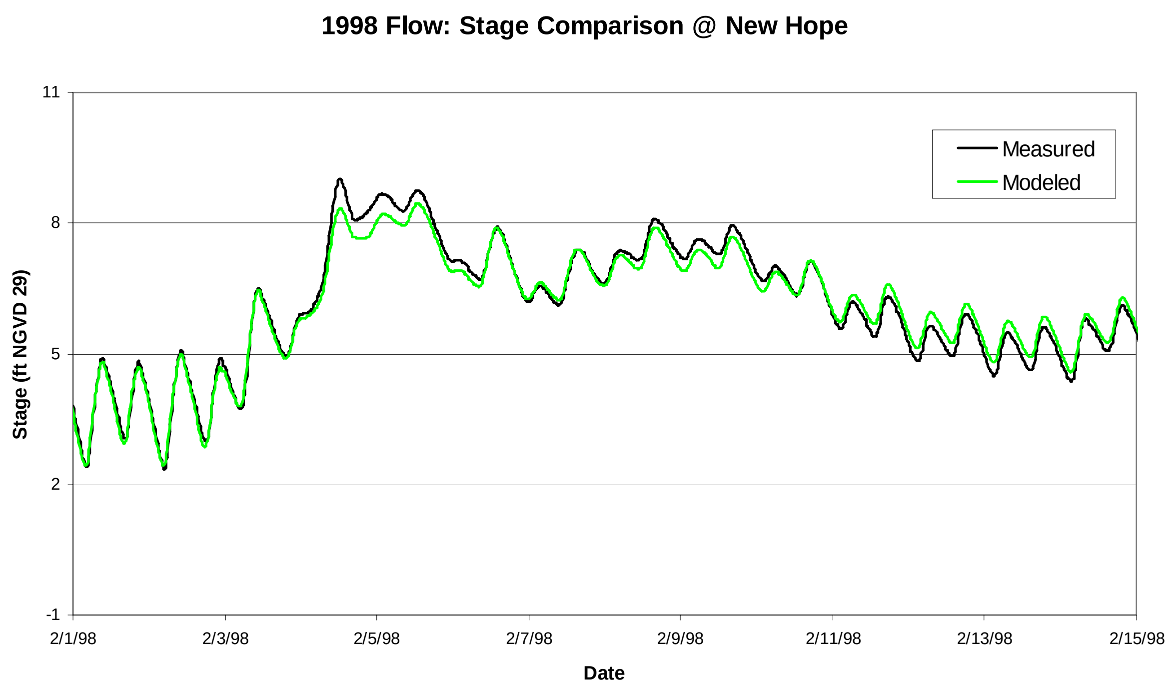
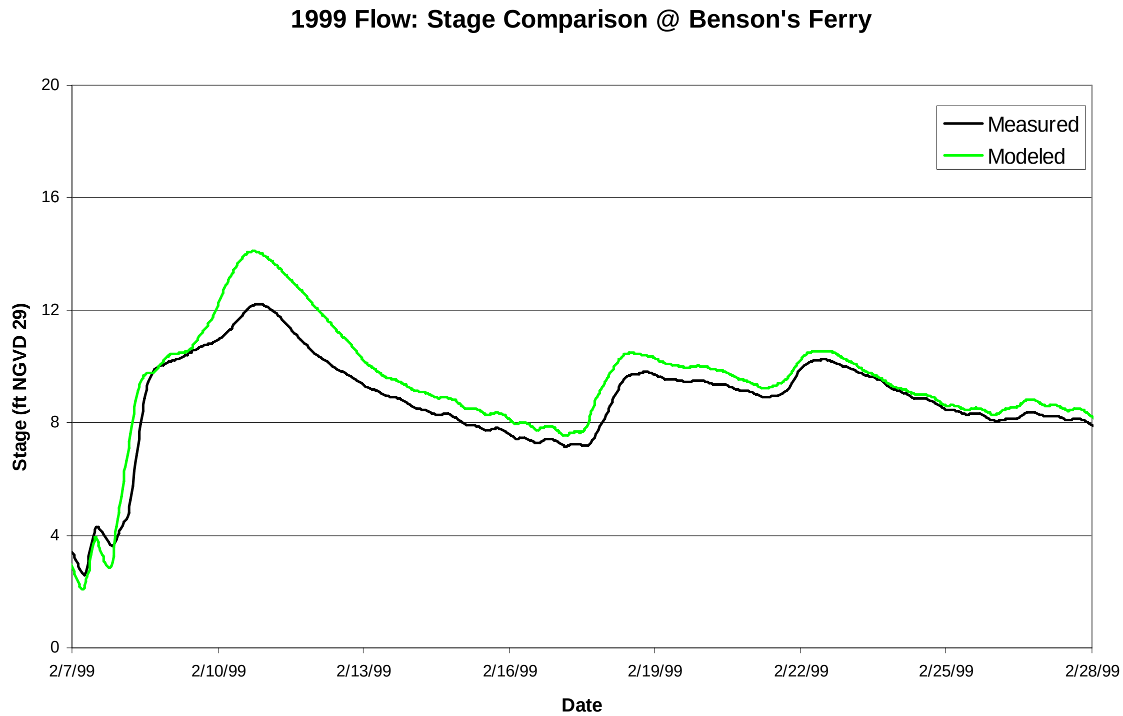


Figure E-8 Model results compared to measured data at Benson’s Ferry (top panel) and New Hope (bottom panel) for the year 1998 flow.



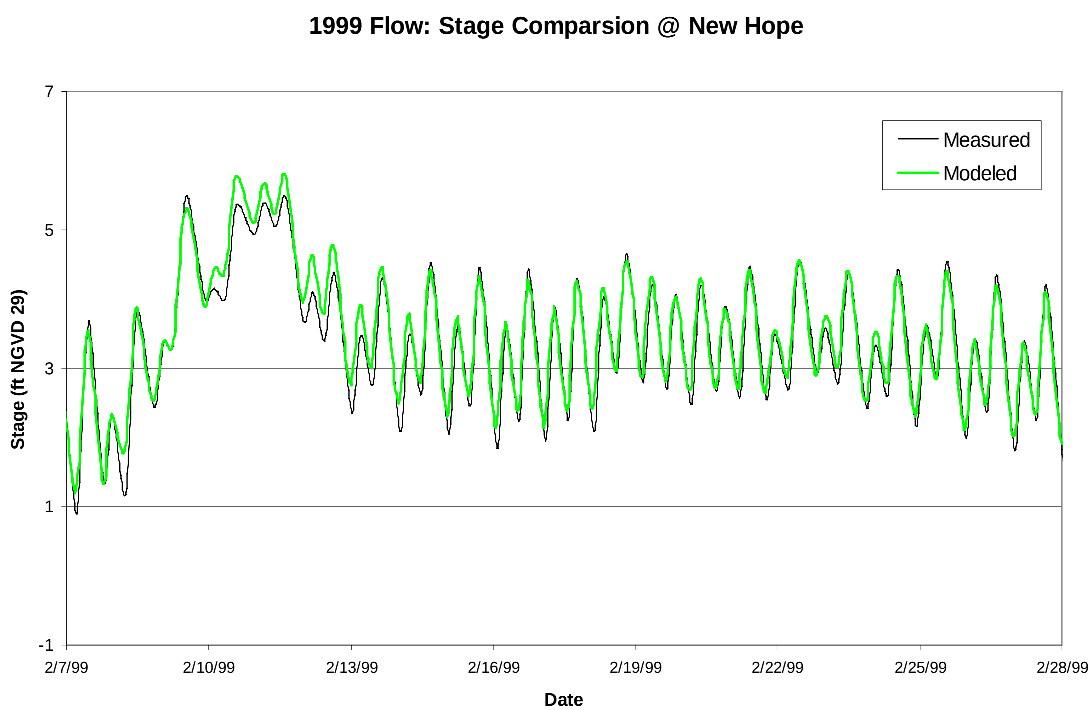
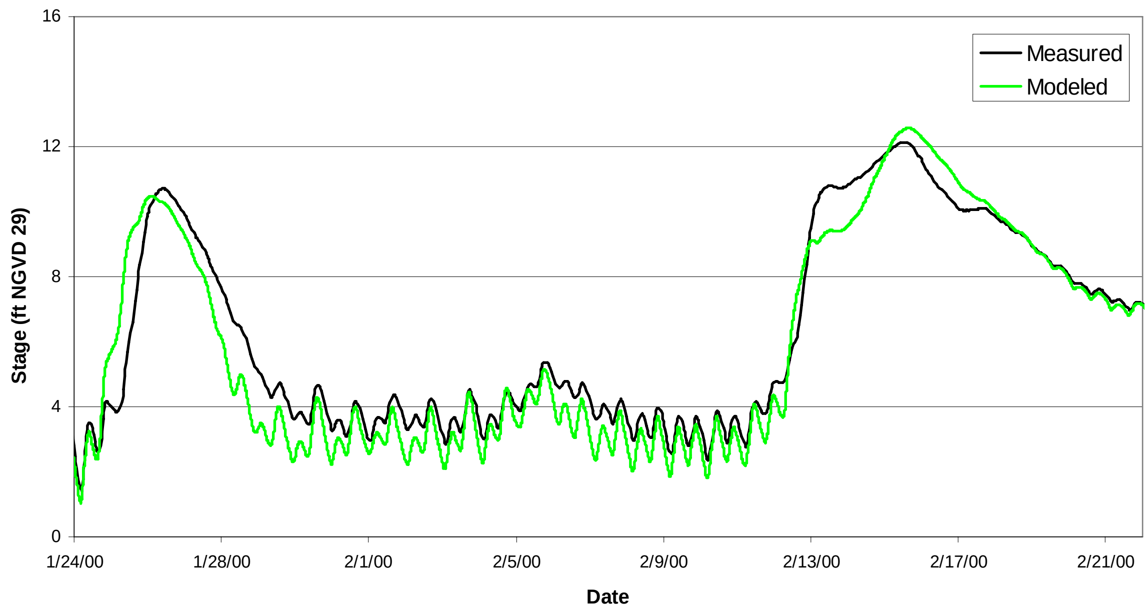


Figure E-9 Model results compared to measured data at Benson’s Ferry (top panel) and New Hope (bottom panel) for the year 1999 flow.

2000 Flow: Stage Comparison @ Benson's Ferry



2000 Flow: Stage Comparison @ New Hope

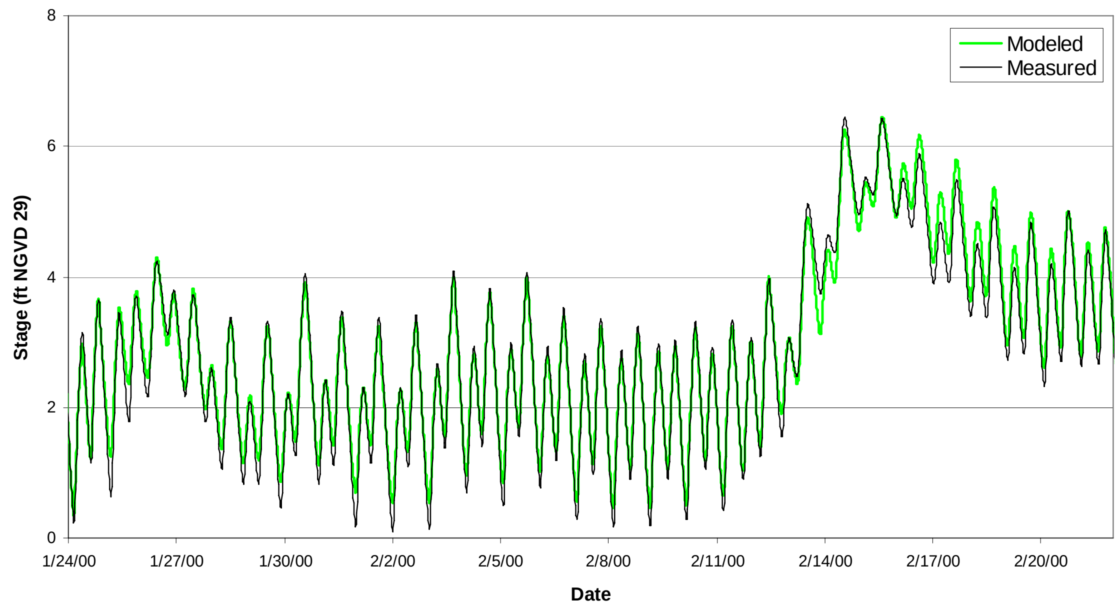


Figure E-10 Model results compared to measured data at Benson’s Ferry (top panel) and New Hope (bottom panel) for the year 2000 flow.

Sensitivity Analysis

To determine the sensitivity of the model's results to various input parameters, sensitivity runs were performed. In conducting a sensitivity analysis, one input parameter was adjusted while all other parameters were left unchanged. The model sensitivity to three types of input parameters were investigated:

- The timing and magnitude of upstream discharge (Cosumnes River at Michigan Bar, Dry Creek above Galt, Mokelumne River at Woodbridge and the Sacramento River at Georgiana Slough),
- Downstream water level (Mokelumne River at Georgiana Slough and Little Potato Slough near Terminous Tract), and
- Channel roughness.

The first four months of flow in 1998 (1/3/98 to 4/30/98) were chosen for the sensitivity analysis, to allow for the analysis of tidally dominated/low river flow conditions in addition to flood events of varying magnitude (up to ~10 year return interval at Michigan Bar). The sensitivity analysis indicated that the model was sensitive to alterations of most input parameters, with varying degrees of sensitivity observed at Benson's Ferry and New Hope Landing.

Levee Failure Criteria

Levee failures have a significant influence upon water levels in the North Delta. Many levee failures occurred during the floods of 1986 and 1997, which impacted the water surface elevations in the channels and inundated adjacent lands. Reasonably good data exists for the levee failures that occurred during the 1986 and 1997 floods. Therefore, it was possible to calibrate the model for these events. Historic levee breaks from these floods were triggered in the model by water surface elevation. Breach dimensions were estimated based on the data available. However, further consideration was required regarding the potential for other levee failures when the system was modified to simulate Alternatives.

Regardless of the methods used to develop levee failure criteria, there was much uncertainty when predicting a levee failure due to high water levels. The Department of Water Resources, in coordination with the North Delta Improvements Group, adopted systematic levee failure criteria for the North Delta MIKE11 model. Levee failure criteria were developed for river reaches west of Interstate 5 based on existing North Delta area breach data. Due to lack of topographic data in many areas on the upper and lower Cosumnes River east of I-5, historic breaks were simulated along these reaches in the model for all model 1997 runs. Because the magnitude of the 1997 event was large and the levees along the Cosumnes are very low and expected to overtop in large events, this was deemed a reasonable assumption.

Lateral flow due to levee overtopping allows for exchange of flow between floodplain conveyance and the river channel. Floodwater enter the overbank areas by overtopping and breaching the levee structure. The rate of levee overflow was computed by the broad-crested weir relationship. The model has the capability to compute flow through breached levees. Input parameters were the failure mode, final bottom width, final bottom elevation, left slope, right slope, and final formation time.

Breach locations were identified by determining the point on each river reach where the distance from the top of the levee (from topographic data) and the maximum water surface elevation (from 1997 base condition MIKE11 runs) was minimum. The failure mode was by overtopping. The final breach dimensions and other parameters are as follows:

- Final bottom width: 500 feet (recommendation from General Characterization of Unplanned Levee Breach Geometries – DWR)

- Breach depth: 40 feet (recommendation from General Characterization of Unplanned Levee Breach Geometries – DWR)

- Final bottom elevation: Existing ground surface elevation on landside of levee

- Left slope: 1

- Right slope: 1

- Model breach as a broad crested weir with weir coefficient of 2.6 (coefficient varies between 2.6 and 3.1 depending on levee cross sectional characteristics – Skogerboe and Hyatt, 1967)

- Rate of breach formation: 1 ft/hr (Powledge et al. 1989)

Flood Control and Ecosystem Restoration Alternatives Modeling

Hydraulic modeling of the North Delta area over a wide range of flows was performed to characterize the current system hydraulically, and to comparatively evaluate the potential impacts of flood control and ecosystem restoration Project Alternatives. The following list includes the hydrologic events and simulation periods for the modeling results presented in this section.

Table E-5 Simulation period and return interval of hydrology

Year	Simulation Period	Return Interval[1]
2000	1/3/2000 till 4/30/2000	~2.5
1999	1/3/1999 till 4/30/1999	~5
1998	1/3/1998 till 4/30/1998	~10
1986	1/3/1986 till 4/30/1986	~25
1997	12/3/1996 till 1/15/1997	200+

[1] Return interval for annual peak flow at Michigan Bar gage on Cosumnes River.

Comparative Simulations for Alternatives

Simulations of Project Alternatives were performed for the flood events listed in Table E-5 and for a 100-yr flood event. Early modeling runs established that there were no appreciable differences between the various flood control and ecosystem restoration configurations on McCormack-Williamson Tract (Group 1 Actions as described in Chapter 2) with regard to system-wide flood performance. This is because all the scenarios on McCormack-Williamson Tract include lowering the East levee to 8.5 ft (NGVD 29) which is the greatest significant flood performance control in the area. Therefore, the Group 2 Alternatives were run with Ecosystem option #2 (i.e., Alternative 1-B) only, and this was taken as representative of performance of any of the McCormack-Williamson Tract Group 1 options in combination with the modeled Group 2 component.

For the purpose of displaying the modeling results in this Appendix, the following naming conventions are used in the Tables and Figures herein. Detailed descriptions of the components of each Alternative are provided in Chapter 2 of the EIR.

Eco-Scenario #2 = Alternative 1-B or Seasonal Floodplain Optimization

Flood Option #1 = Alternative 2-A or North Staten Detention

Flood Option #2 = Alternative 2-B or West Staten Detention

Flood Option #3 = Alternative 2-C or East Staten Detention

Flood Option #4 = Alternative 2-D or Dredge and Levee Modifications

The results of the flood control modeling are presented in several ways. The maximum stage at each of the model index points for each of the runs are shown in Table E-6 for 1986 hydrology, Table E-7 for the 1997 hydrology, and Table E-8 for the 100-yr flood hydrology. Stage hydrographs are shown in Figures E-11 through E-30 at representative points including New Hope, Benson's Ferry, and downstream locations on the North and South Forks of the Mokelumne for the 1997 hydrology. The plots are focused in the time windows where noticeable changes were observed. These provide a comparison of stage duration with and without the Project Alternative. A full set of stage hydrographs at each index point for each modeled hydrology can be made available on CD by request.

Table E-9 provides a comparison of maximum velocities at key points for each of the flood control Alternatives (combined with Alternative 1-B, ecological option 2) for 1986 and 1997 hydrology. Figures E-31 and E-32 show flow splits for the North and South Forks of the Mokelumne River for each of the Alternatives for 1986 and 1997 hydrology. South Fork and North fork flows were estimated at approximately 2 miles downstream from the New Hope Bridge and Miller Ferry Bridge, respectively. The flow-split comparisons are intended to provide a rough qualitative idea of how flow-splits may change for each of the Project Alternatives. Of note, because of the complexity of the hydraulic system, the flow splits should be considered in context with the respective stage hydrographs, detention basin volumes, and other flows throughout the system. For example, there is not necessarily a direct correlation between volumes captured in Staten detention basins and instantaneous flow remaining in the North and South Forks.

Table E-6 Comparison of Group 2 Project Alternatives: Water Level Impacts for 1986 Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)						
		1986 Flood	1986 No Failures	Alternative 1-B (Base Case)	Group 2 Alternatives, Combined with Alternative 1-B			
					Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
BF-1	Benson's Ferry	17.8	18.8	16.3 (2.5)[1]	15.6 (3.2)	15.8 (3.0)	15.8 (3.0)	15.5 (3.3)
MR-2	Mokelumne River	14.4	15.6	13.6 (2.0)	11.6 (4.0)	12.5 (3.1)	12.6 (3.0)	12.1 (3.5)
SG-3	Snodgrass Slough	12.9	15.0	14.3 (0.7)	12.7 (2.3)	13.4 (1.6)	13.5 (1.5)	13.0 (2.0)
NH-4	New Hope	12.5	13.3	13.3 (0)	11.0 (2.3)	12.1 (1.2)	12.2 (1.1)	12.0 (1.3)
SF-5	SF[2] Mokelumne	8.7	9.4	9.3 (0.1)	8.2 (1.2)	8.7 (0.7)	8.3 (1.1)	9.1 (0.3)
SF-6	SF Mokelumne	7.2	7.6	7.6 (0)	7.2 (0.4)	7.3 (0.3)	7.2 (0.4)	7.9 (-0.3)
SF-7	SF Mokelumne	6.9	7.3	7.3 (0)	7.0 (0.3)	7.1 (0.2)	7.0 (0.3)	7.4 (-0.1)
NF-8	NF Mokelumne	11.3	12.5	12.7 (-0.2)	10.8 (1.7)	11.2 (1.3)	11.7 (0.8)	11.5 (1.0)
NF-9	NF Mokelumne	8.4	9.6	9.7 (-0.1)	8.6 (1.0)	8.8 (0.8)	9.1 (0.5)	9.0 (0.6)
NF-10	NF Mokelumne	6.9	7.9	7.9 (0)	7.4 (0.5)	7.5 (0.4)	7.6 (0.3)	7.7 (0.2)
MC-11	McConnell	46.3	46.3	46.3 (0)	46.2 (0.1)	46.2 (0.1)	46.2 (0.1)	46.3 (0)
TC-12	Twin Cities Road	24.9	24.9	24.7 (0.2)	24.6 (0.3)	24.6 (0.3)	24.6 (0.3)	24.7 (0.2)
LR-13	Lambert Road	12.9	15.0	14.3 (0.7)	12.7 (2.3)	13.4 (1.6)	13.5 (1.5)	13.0 (2.0)
PP-14	Point Pleasant	13.5	13.9	13.5 (0.4)	11.2 (2.7)	13.4 (0.5)	13.4 (0.5)	13.4 (0.5)

Index Point	Location	Peak Stage (ft NGVD 29)						
					Group 2 Alternatives, Combined with Alternative 1-B			
		1986 Flood	1986 No Failures	Alternative 1-B (Base Case)	Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
TT-15	Terminous Tract	6.8	7.1	7.2 (-0.1)	6.9 (0.2)	7.0 (0.1)	7.0 (0.1)	7.2 (-0.1)
NS-16	Confluence of NF and SF	6.8	7.2	7.2 (0)	7.0 (0.2)	7.0 (0.2)	7.0 (0.2)	7.2 (0)
Detention basin volume (ac-ft)					48,300[3]	35,600[4]	32,400[4]	N/A

[1] Value in parentheses denotes: stage difference (ft) = Stage for "No Failure" – Stage for "Alternative";

Positive value denotes <u>stage drop.</u>

[2] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

[3] 10-ft weir height

[4] 9-ft weir height

Table E-7 Comparison of Group 2 Project Alternatives: Water Level Impacts for 1997 Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)						
					Group 2 Alternatives, Combined with Alternative 1-B			
		1997 Flood	1997 No Failures	Alternative 1-B (Base Case)	Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
BF-1	Benson's Ferry	19.2	19.9	17.4 (2.5)[1]	16.8 (3.1)	17.2 (2.7)	17.1 (2.8)	16.6 (3.3)
MR-2	Mokelumne River	16.1	16.9	14.6 (2.3)	12.1 (4.8)	13.3 (3.6)	13.6 (3.3)	12.9 (4.0)
SG-3	Snodgrass Slough	15.0	16.3	15.4 (0.9)	13.9 (2.4)	14.4 (1.9)	14.7 (1.6)	13.8 (2.5)
NH-4	New Hope	14.3	14.5	14.3 (0.2)	11.4 (3.1)	12.7 (1.8)	13.1 (1.4)	12.8 (1.7)
SF-5	SF[2] Mokelumne	9.6	9.7	9.7 (0)	7.9 (1.8)	8.7 (1.0)	8.2 (1.5)	9.3 (0.4)
SF-6	SF Mokelumne	7.2	8.3	7.2 (1.1)	6.4 (1.9)	6.7 (1.6)	6.6 (1.7)	7.6 (0.7)
SF-7	SF Mokelumne	6.7	6.8	6.7 (0.1)	6.2 (0.6)	6.4 (0.4)	6.3 (0.5)	6.9 (-0.1)
NF-8	NF Mokelumne	13.4	13.6	13.6 (0)	11.1 (2.5)	11.5 (2.1)	12.7 (0.9)	12.2 (1.4)
NF-9	NF Mokelumne	9.9	10.0	10.1 (-0.1)	8.4 (1.6)	8.8 (1.2)	9.4 (0.6)	9.2 (0.8)
NF-10	NF Mokelumne	7.7	7.8	7.8 (0)	6.9 (0.9)	7.1 (0.7)	7.4 (0.4)	7.4 (0.4)
MC-11	McConnell	49.8	49.8	49.8 (0)	49.7 (0.1)	49.7 (0.1)	49.7 (0.1)	49.8 (0)
TC-12	Twin Cities Road	25.8	25.8	25.6 (0.2)	25.6 (0.2)	25.6 (0.2)	25.6 (0.2)	25.6 (0.2)
LR-13	Lambert Road	15.0	16.3	15.4 (0.9)	13.9 (2.4)	14.4 (1.9)	14.7 (1.6)	13.8 (2.5)

Index Point	Location	Peak Stage (ft NGVD 29)						
		1997 Flood	1997 No Failures	Alternative 1-B (Base Case)	Group 2 Alternatives, Combined with Alternative 1-B			
					Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
PP-14	Point Pleasant	12.5	12.7	12.5 (0.2)	12.3 (0.4)	12.4 (0.3)	12.5 (0.2)	12.5 (0.2)
TT-15	Terminous Tract	6.5	6.5	6.5 (0)	6.0 (0.5)	6.2 (0.3)	6.2 (0.3)	6.6 (-0.1)
NS-16	Confluence of NF and SF	6.7	6.7	6.7 (0)	6.3 (0.4)	6.4 (0.3)	6.5 (0.2)	6.6 (0.1)
Detention basin volume (ac-ft)					36,900[3]	24,800[4]	21,200[4]	N/A

[1] Value in parentheses denotes: stage difference (ft) = Stage for “No Failure” – Stage for “Alternative”;
Positive value means stage drop.

[2] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

[3] 10-ft weir height

[4] 9-ft weir height

Table E-8 Comparison of Group 2 Project Alternatives: Water Level Impacts for 100-Yr Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)					
				Group 2 Alternatives, Combined with Alternative 1-B			
		100-year No Failures	Alternative 1-B (Base Case)	Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
BF-1	Benson's Ferry	18.7	16.1 (2.6)[1]	15.9 (2.8)	16.0 (2.7)	16.0 (2.7)	15.7 (3.0)
MR-2	Mokelumne River	15.3	13.0 (2.3)	12.0 (3.3)	12.5 (2.8)	12.6 (2.7)	11.8 (3.5)
SG-3	Snodgrass Slough	14.6	13.8 (0.8)	11.5 (3.1)	13.4 (1.2)	13.5 (1.1)	12.2 (2.4)
NH-4	New Hope	12.9	12.8 (0.1)	11.5 (1.4)	12.2 (0.7)	12.3 (0.6)	11.7 (1.2)
SF-5	SF[2] Mokelumne	8.7	8.5 (0.2)	7.9 (0.8)	8.2 (0.5)	8.1 (0.6)	8.5 (0.2)
SF-6	SF Mokelumne	6.9	6.9 (0)	6.7 (0.2)	6.8 (0.1)	6.8 (0.1)	7.2 (-0.3)
SF-7	SF Mokelumne	6.7	6.7 (0)	6.5 (0.2)	6.6 (0.1)	6.6 (0.1)	6.8 (-0.1)
NF-8	NF Mokelumne	12.1	12.1 (0)	11.2 (0.9)	11.2 (0.9)	11.7 (0.4)	11.2 (0.9)
NF-9	NF Mokelumne	8.9	8.8 (0.1)	8.4 (0.5)	8.5 (0.4)	8.6 (0.3)	8.4 (0.5)
NF-10	NF Mokelumne	7.3	7.3 (0)	7.2 (0.1)	7.3 (0)	7.3 (0)	7.1 (0.2)
MC-11	McConnell	48.0	48.0 (0)	48.0 (0)	48.0 (0)	48.0 (0)	48.0 (0)
TC-12	Twin Cities Road	25.5	25.4 (0.1)	25.4 (0.1)	25.4 (0.1)	25.4 (0.1)	25.4 (0.1)
LR-13	Lambert Road	14.6	13.8 (0.8)	13.1 (1.5)	13.4 (1.2)	13.5 (1.1)	12.5 (2.1)
PP-14	Point Pleasant	11.9	11.8 (0.1)	11.8 (0.1)	11.8 (0.1)	11.8 (0.1)	11.7 (0.2)
TT-15	Terminous Tract	6.5	6.5 (0)	6.4 (0.1)	6.5 (0)	6.5 (0)	6.6 (-0.1)
NS-16	Confluence of NF and SF	6.8	6.8 (0)	6.7 (0.1)	6.7 (0.1)	6.7 (0.1)	6.7 (0.1)
Detention basin volume (ac-ft)				23,400[3]	16,000[4]	16,100[4]	N/A

[1] Value in parentheses denotes: stage difference (ft) = Stage for “No Failure” – Stage for “Alternative”;

Positive value denotes stage drop.

[2] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

[3] 10-ft weir height

[4] 9-ft weir height

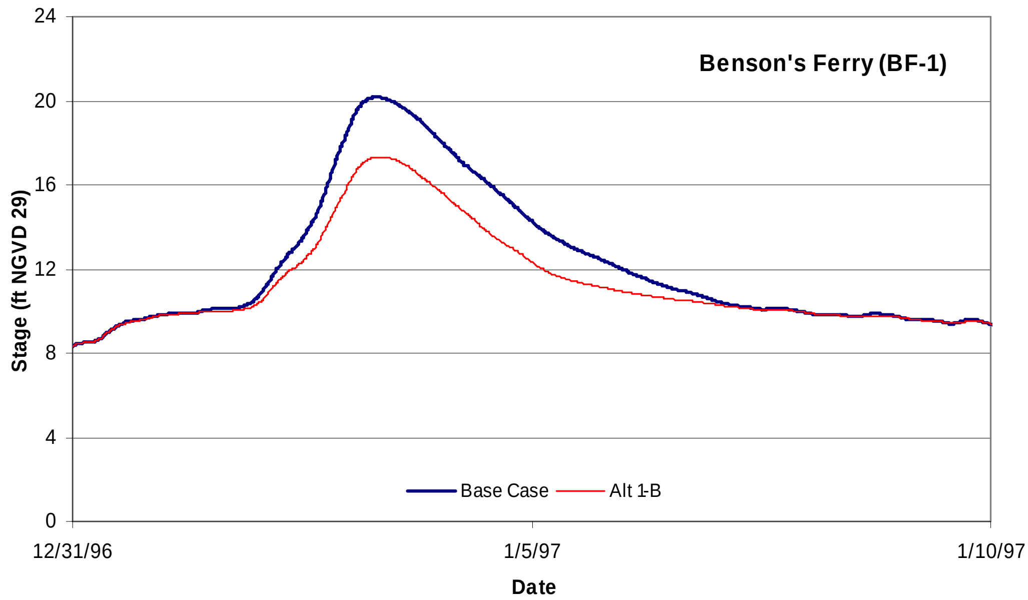


Figure E-11 Model results at Benson’s Ferry for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 1-B and the Base Case (Alternative NP).

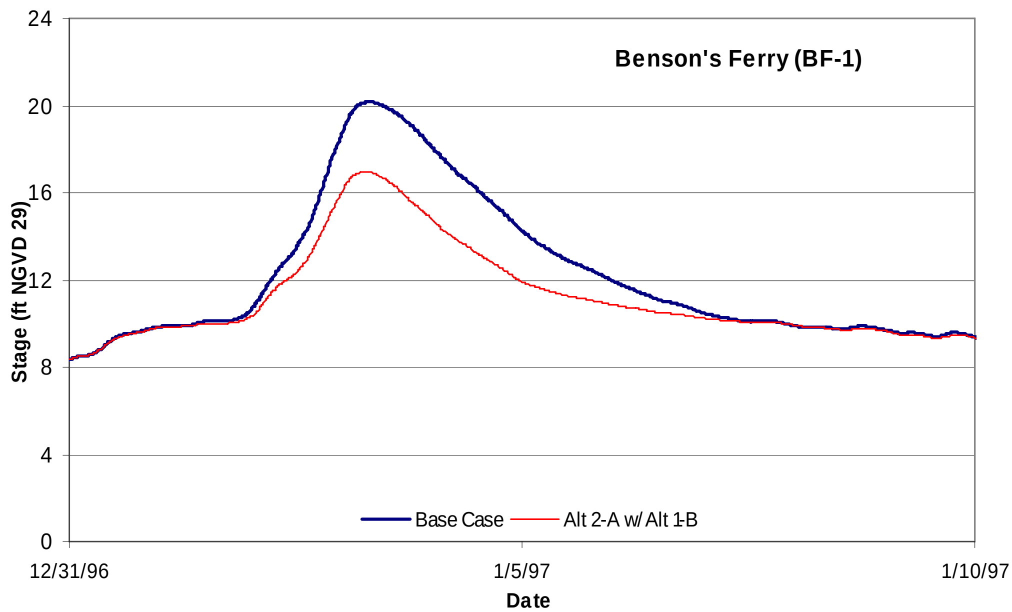


Figure E-12 Model results at Benson’s Ferry for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-A w/ 1-B and the Base Case (Alternative NP).

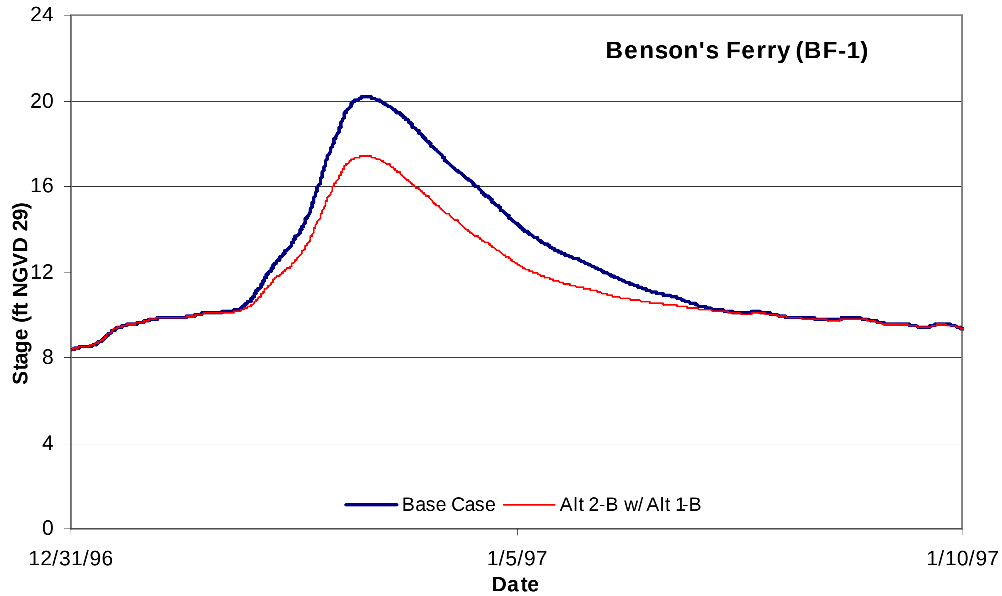


Figure E-13 Model results at Benson's Ferry for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-B w/ 1-B and the Base Case (Alternative NP).

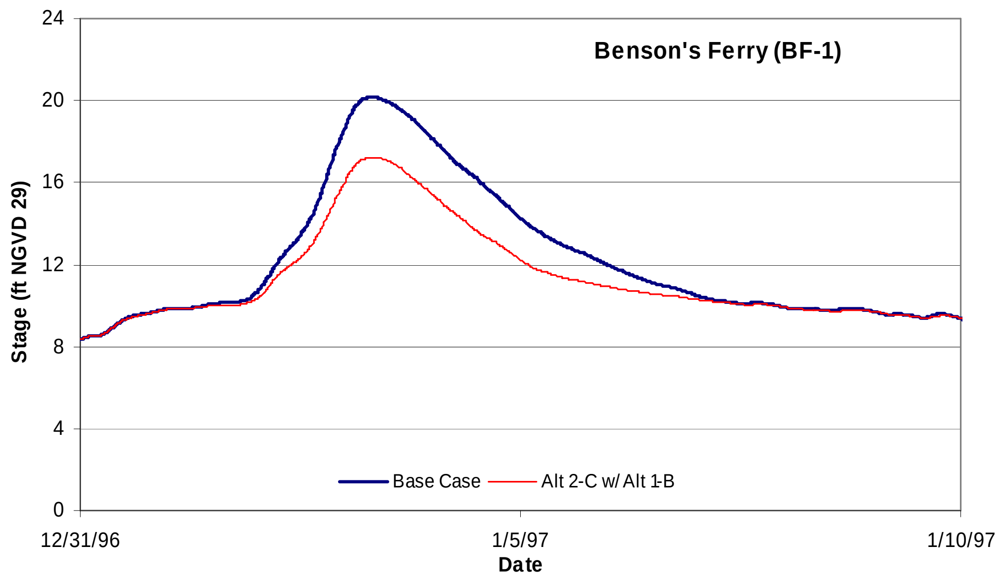


Figure E-14 Model results at Benson's Ferry for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-C w/ 1-B and the Base Case (Alternative NP).

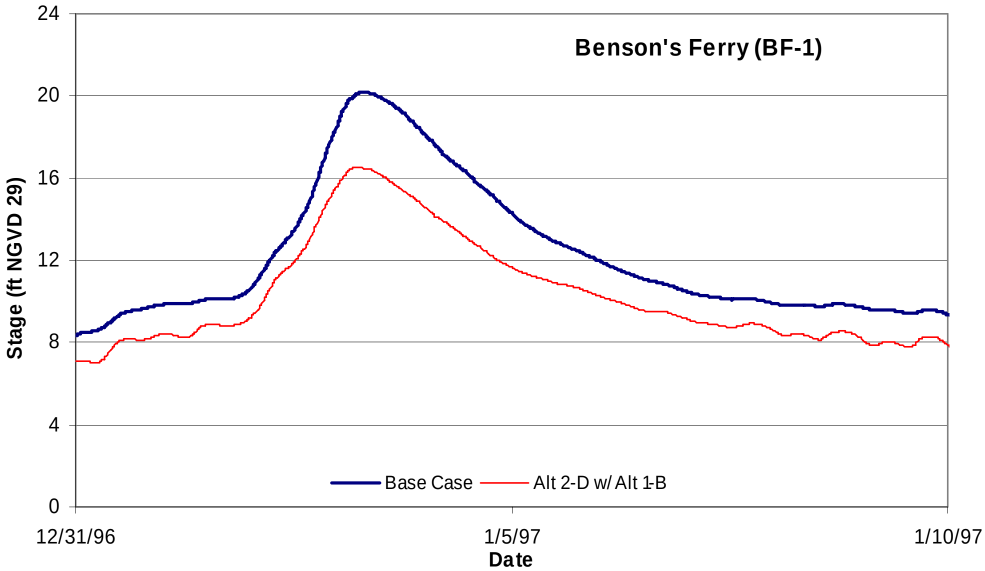


Figure E-15 Model results at Benson’s Ferry for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-D w/ 1-B and the Base Case (Alternative NP).

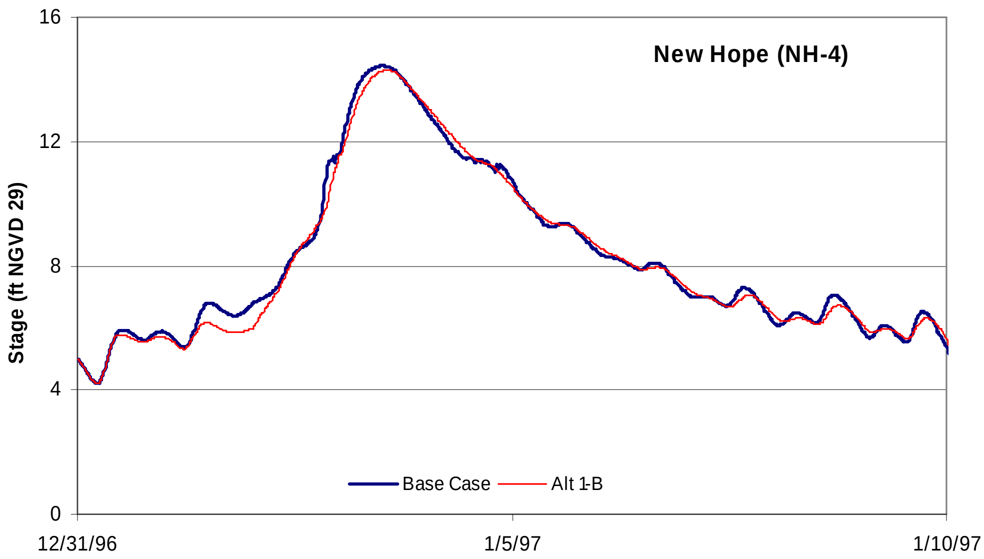


Figure E-16 Model results at New Hope for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 1-B and the Base Case (Alternative NP).

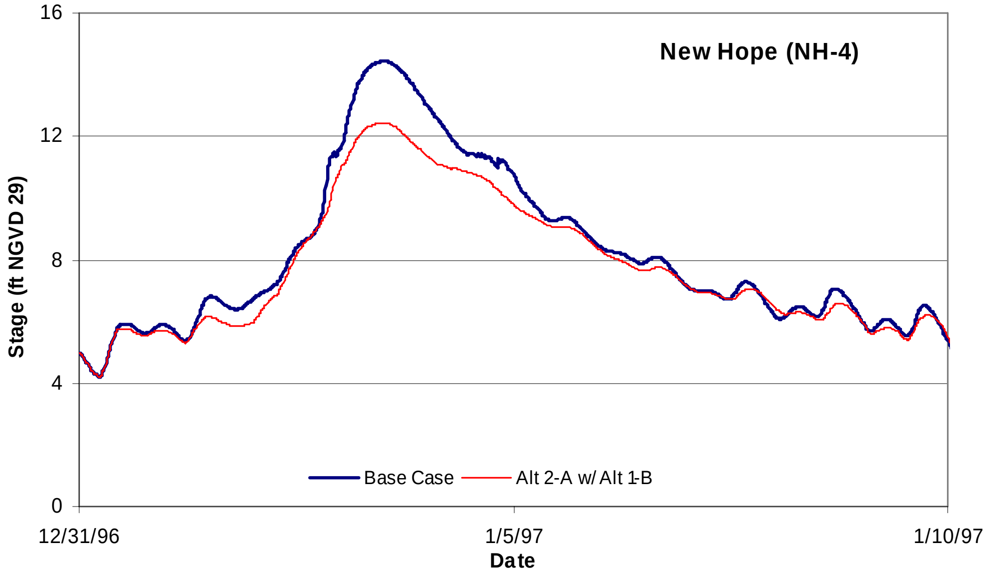


Figure E-17 Model results at New Hope for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-A w/ 1-B and the Base Case (Alternative NP).

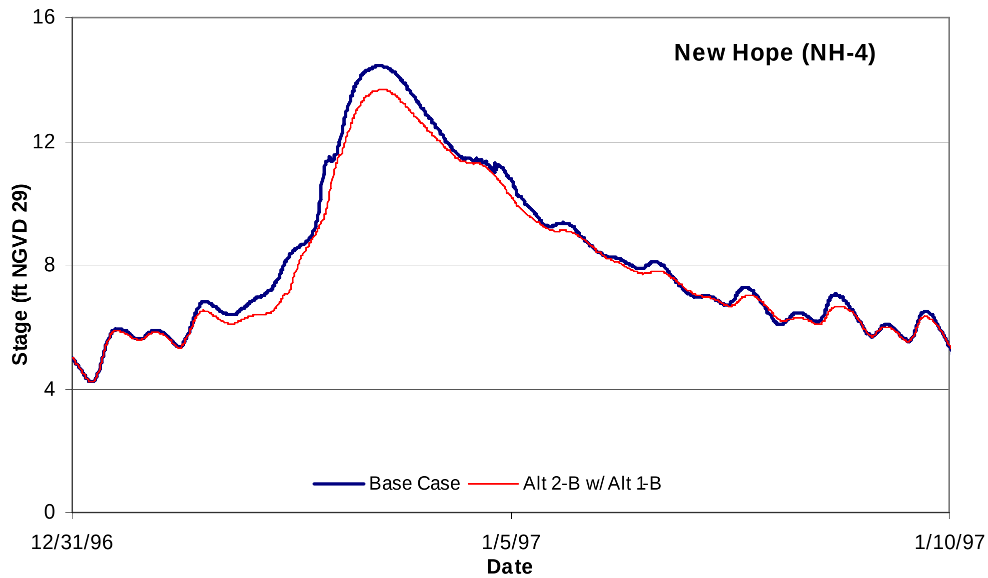


Figure E-18 Model results at New Hope for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-B w/ 1-B and the Base Case (Alternative NP).

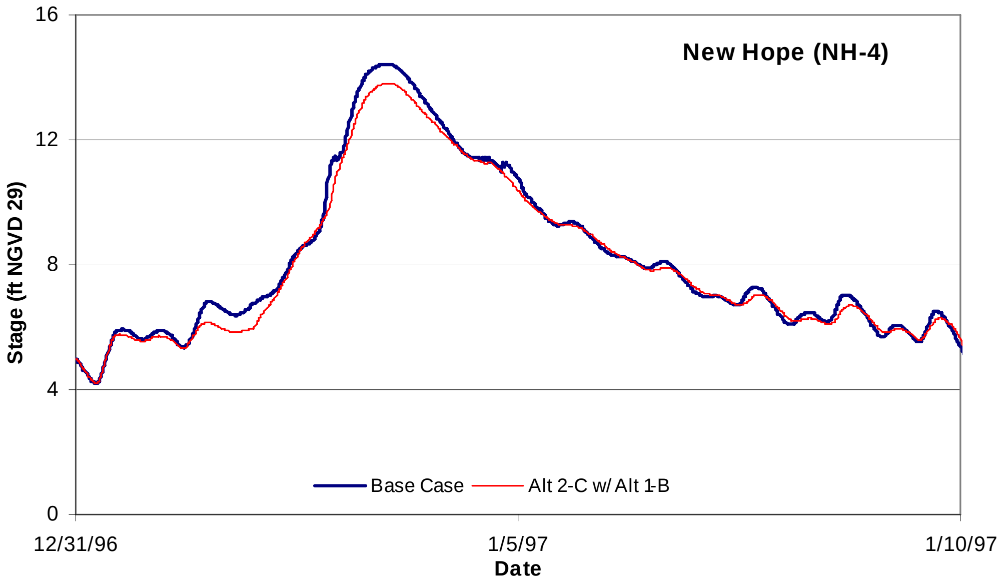


Figure E-19 Model results at New Hope for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-C w/ 1-B and the Base Case (Alternative NP).

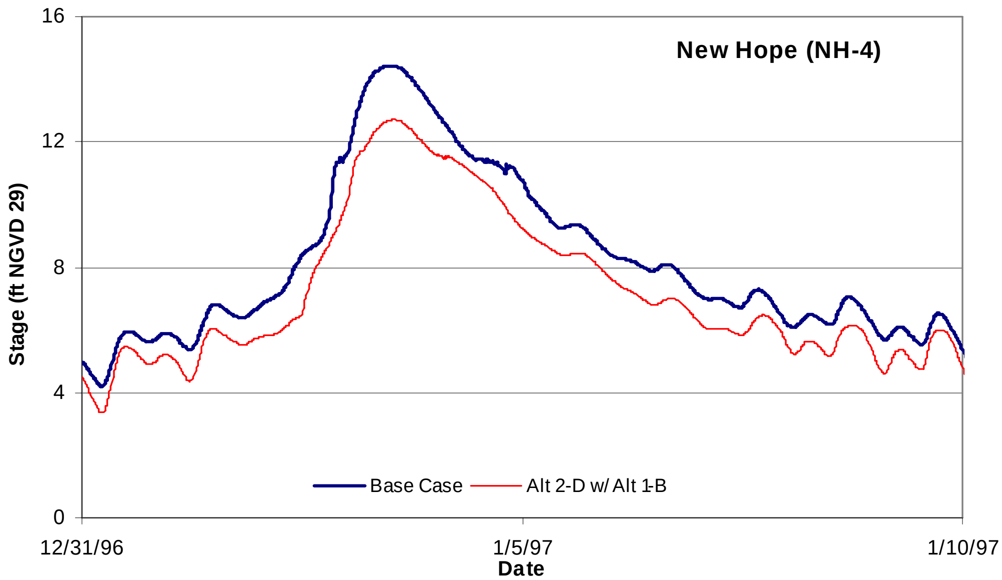


Figure E-20 Model results at New Hope for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-D w/ 1-B and the Base Case (Alternative NP).

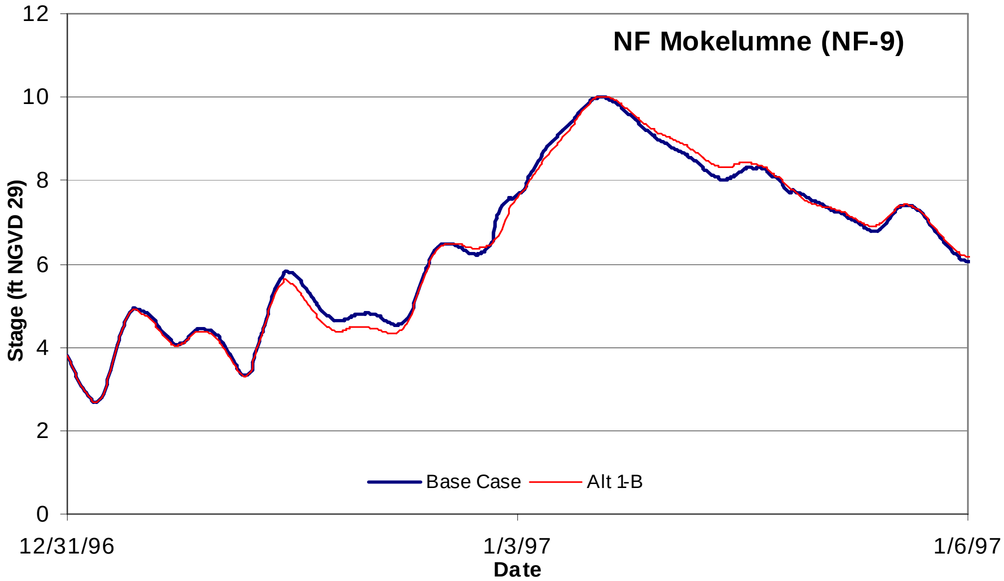


Figure E-21 Model results at NF-9 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 1-B and the Base Case (Alternative NP).

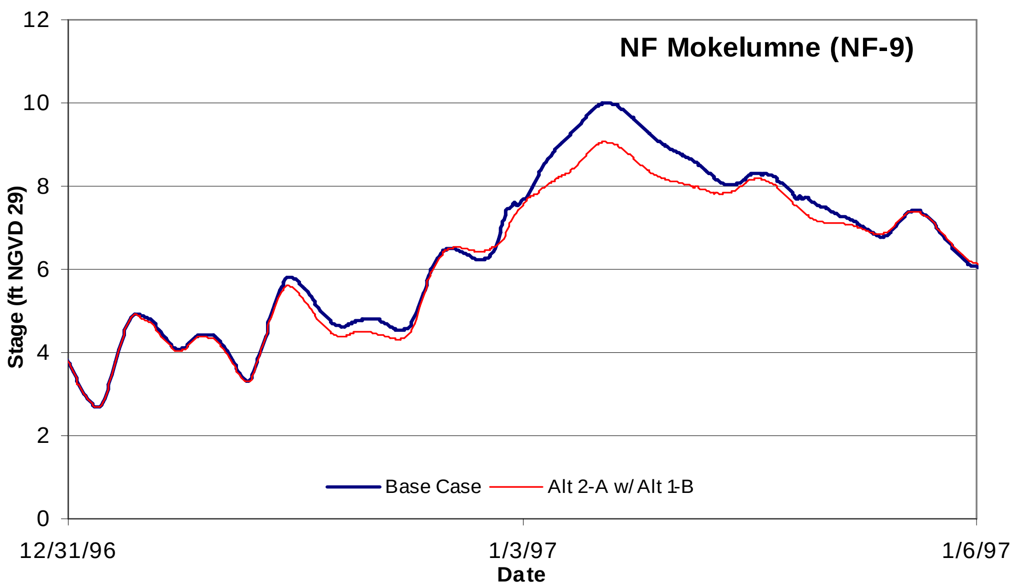


Figure E-22 Model results at NF-9 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-A w/ 1-B and the Base Case (Alternative NP).

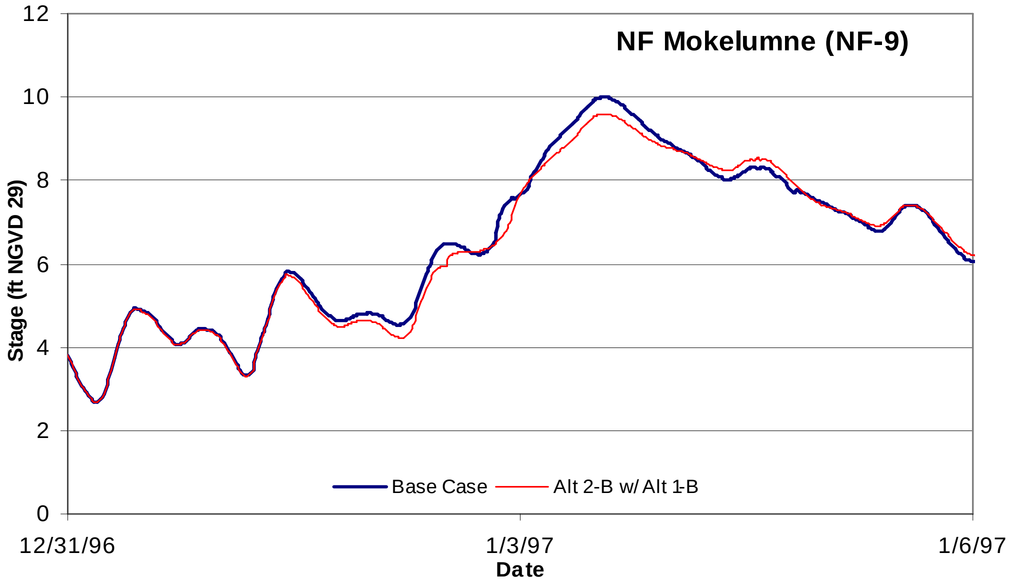


Figure E-23 Model results at NF-9 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-B w/ 1-B and the Base Case (Alternative NP).

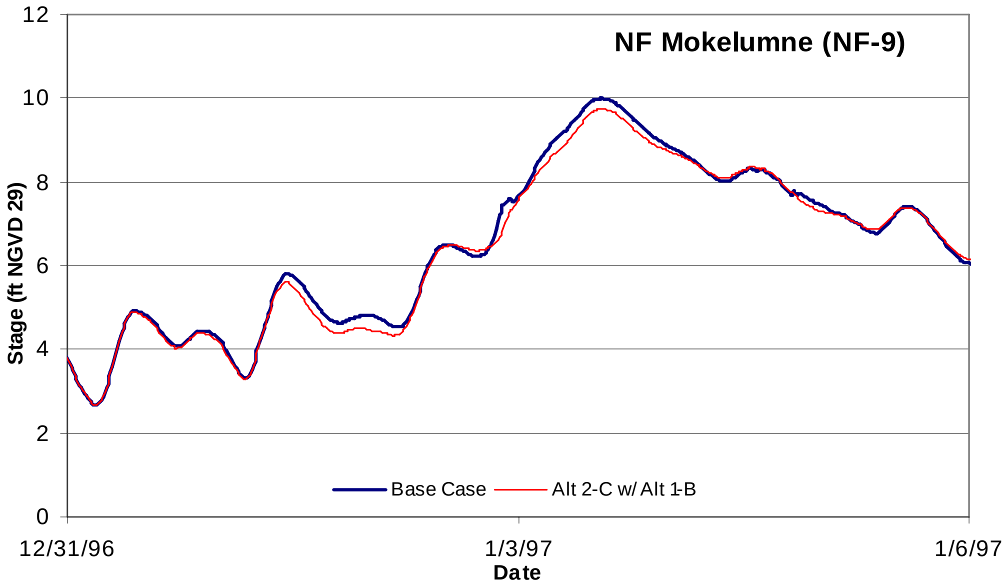


Figure E-24 Model results at NF-9 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-C w/ 1-B and the Base Case (Alternative NP).

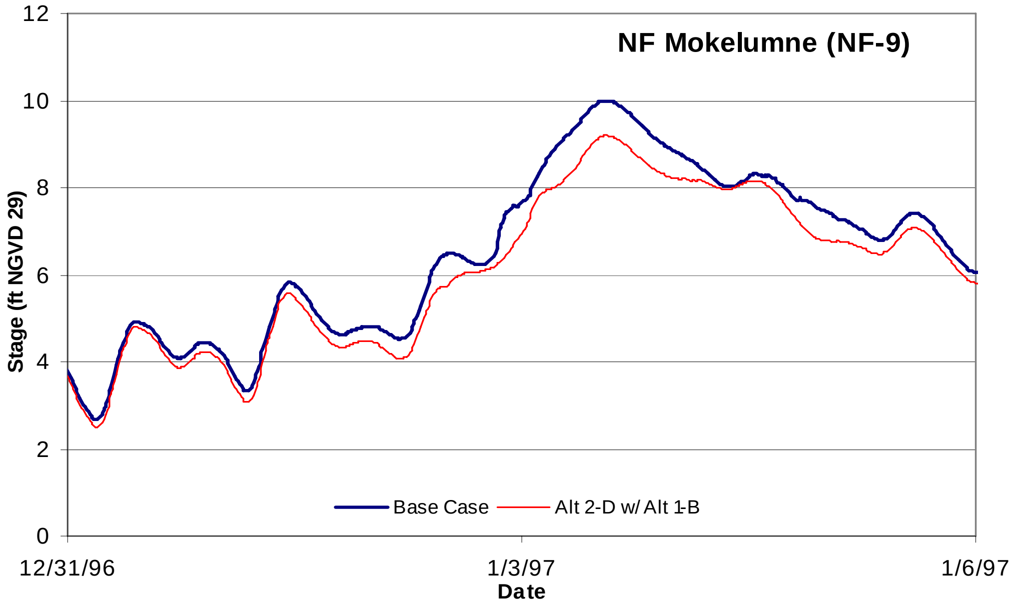


Figure E-25 Model results at NF-9 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-D w/ 1-B and the Base Case (Alternative NP).

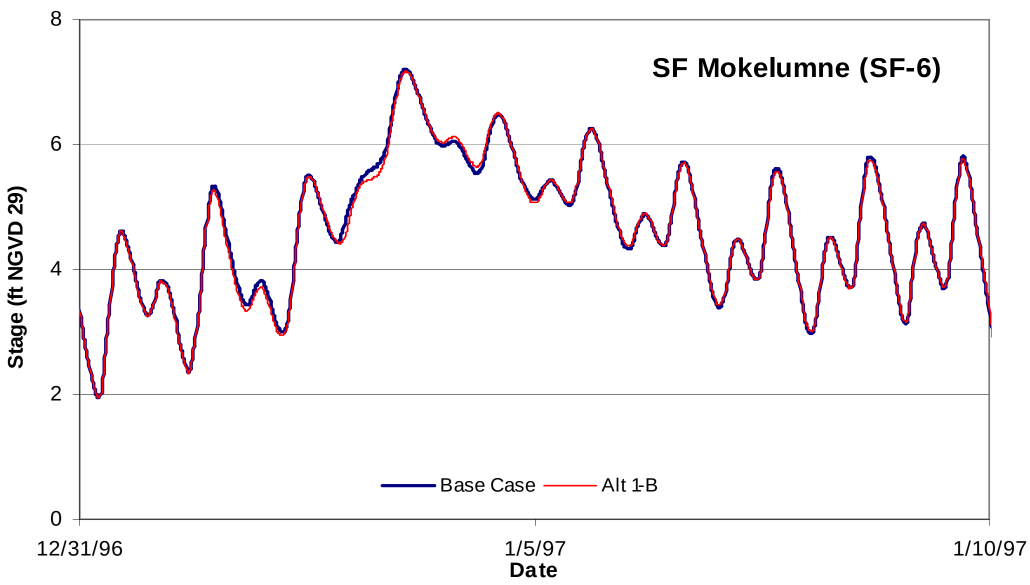


Figure E-26 Model results at SF-6 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between 1-B and the Base Case (Alternative NP).

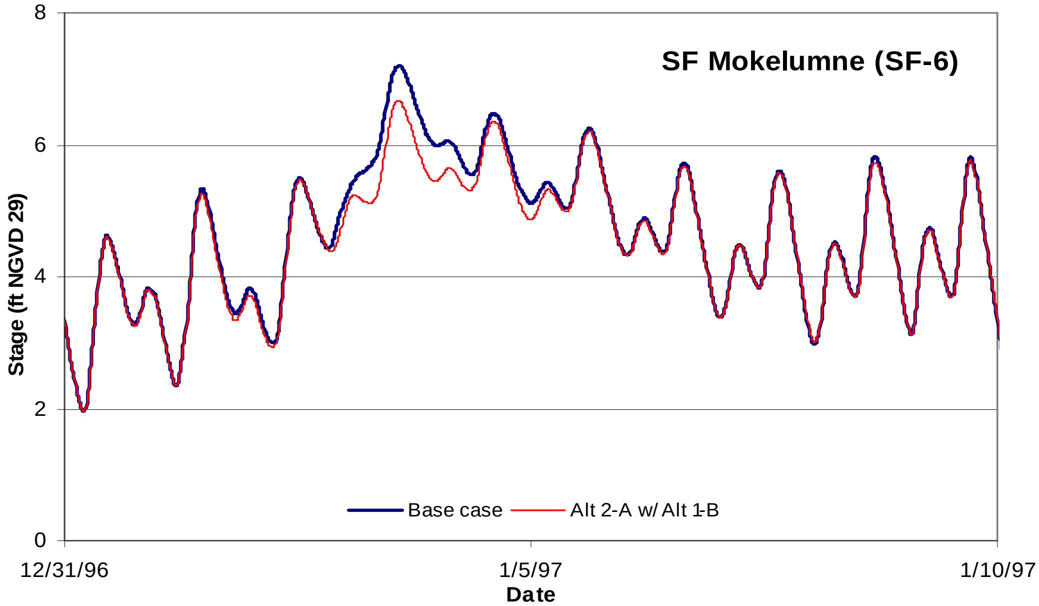


Figure E-27 Model results at SF-6 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-A w/ 1-B and the Base Case (Alternative NP).

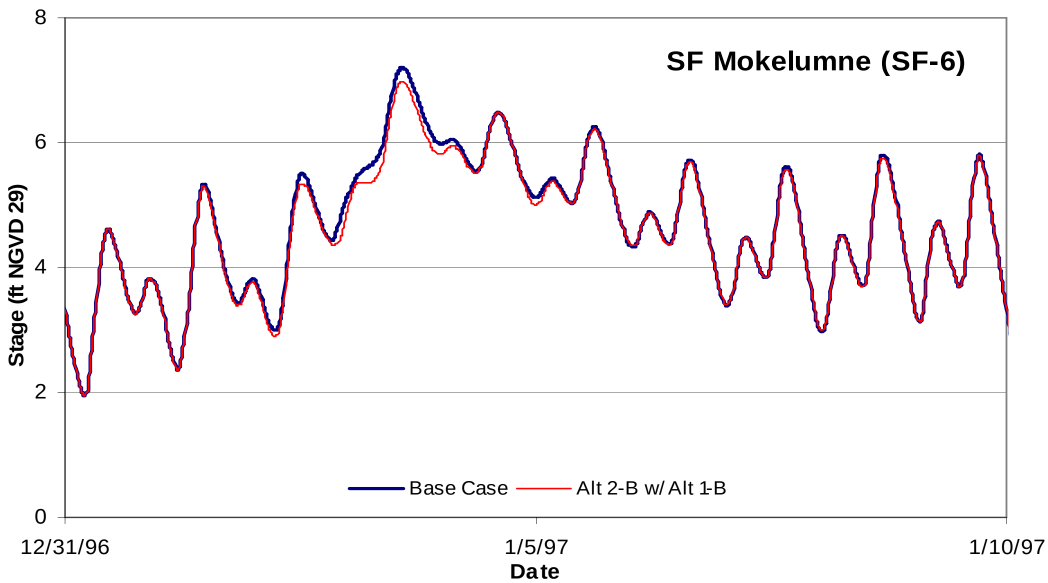


Figure E-28 Model results at SF-6 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-B w/ 1-B and the Base Case (Alternative NP).

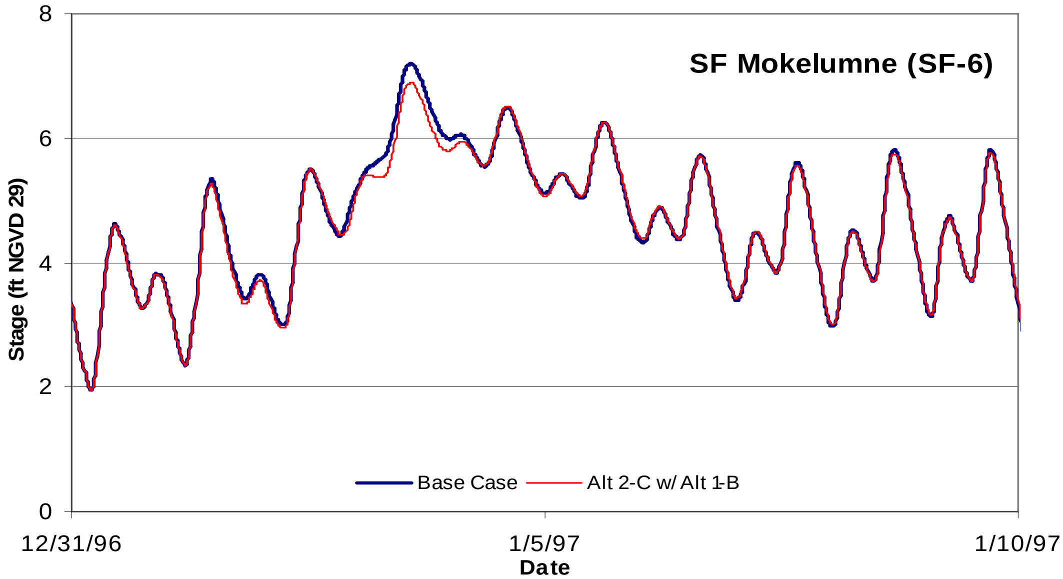


Figure E-29 Model results at SF-6 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-C w/ 1-B and the Base Case (Alternative NP).

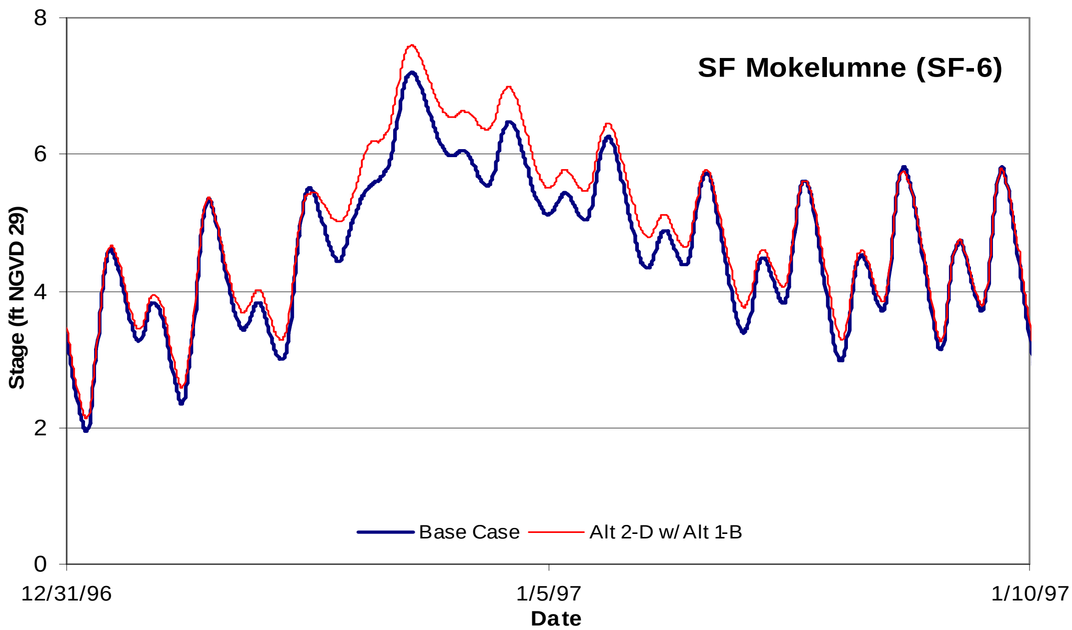


Figure E-30 Model results at SF-6 (for location, see Figure A-5) for the 1997 flood hydrology (with no levee failure): Comparison between Alternative 2-D w/ 1-B and the Base Case (Alternative NP).

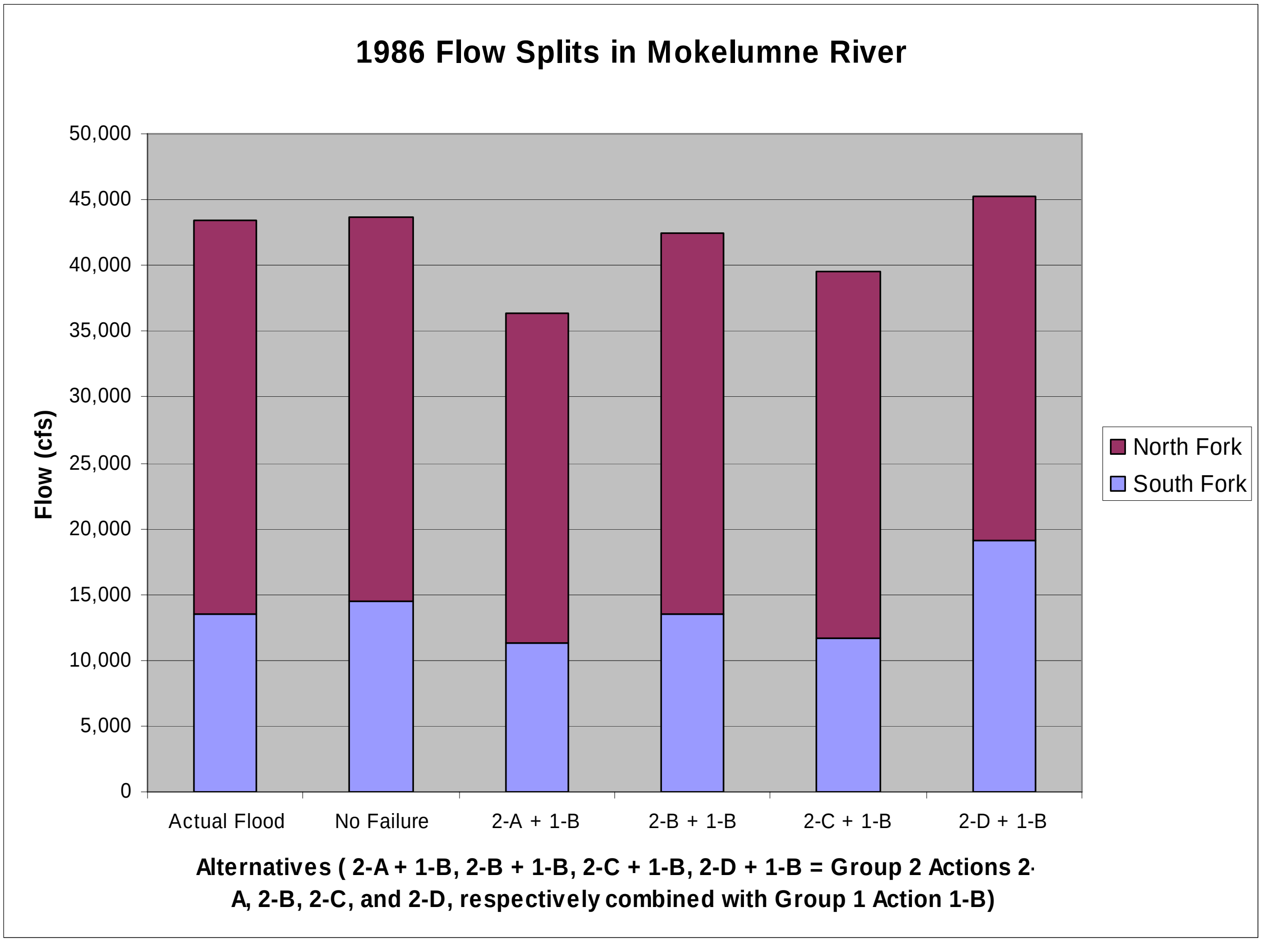


Figure E-31 Flow splits in the South and North Fork of the Mokelumne River for the 1986 flood hydrology.

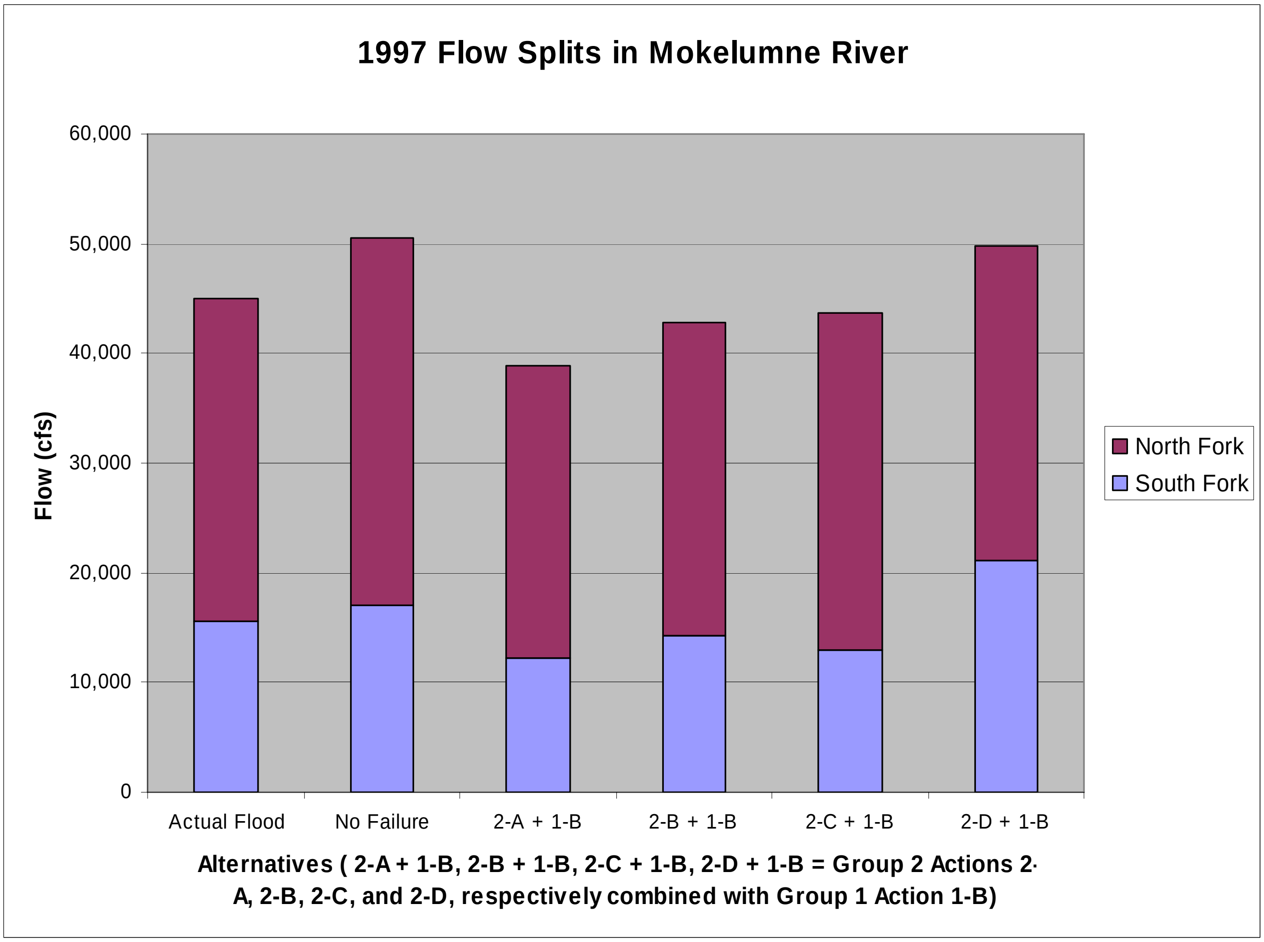


Figure E-32 Flow splits in the South and North Fork of the Mokelumne River for the 1997 flood hydrology.

Table E-9 Comparison of Group 2 Project Alternatives: Maximum Velocities (ft/sec) at Key Points

	1986 Flood						1997 Flood					
			Group 2 Alternatives, Combined with Alternative 1-B						Group 2 Alternatives, Combined with Alternative 1-B			
Index Point[1]	Actual Flood	No Levee Failure	Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D	Actual Flood	No Levee Failure	Alternative 2-A	Alternative 2-B	Alternative 2-C	Alternative 2-D
BF-1	3.2	3.0	3.6	3.6	3.6	3.9	3.0	3.2	3.6	3.4	4.5	3.7
MR-2	4.5	4.6	3.7	3.7	3.6	3.9	5.1	5.1	3.1	3.3	3.1	3.5
NH-4	2.9	2.6	2.6	2.6	2.6	2.2	3.1	2.8	2.8	2.8	2.8	1.9
SF-5	3.9	4.1	3.6	3.9	4.0	4.2	4.8	4.7	4.1	4.5	4.4	4.7
NF-8	5.2	4.9	4.6	5.4	4.8	4.5	5.3	5.4	5.0	5.9	5.2	4.9
NF-9	4.5	4.9	4.6	5.4	4.8	4.5	4.2	4.4	4.1	4.3	4.3	4.0

[1] For Index Point locations, see Figure E-5.

Low Flow Simulations

Simulations of low flows for different Project Alternatives were performed for the 1998, 1999, and the 2000-yr hydrology events. The results of the low flow modeling are presented similarly to the high flow runs. Because the detention basin elements in Alternatives 2-A thru 2-C do not come into play at low flow, only the Group 1 Actions were modeled for the low flow events. The maximum stage at each of the model index points for each of the runs are shown in Table E-10 for 1998 hydrology, Table E-11 for the 1999 hydrology, and Table E-12 for 2000 hydrology.

Stage hydrographs for the 1999 hydrology, are shown in Figures E-33 thru E-43 at representative points including New Hope, Benson's Ferry, and downstream locations on the North and South Forks of the Mokelumne River. The plots are focused in the time windows where changes are observed. These provide a comparison of stage duration with and without the Project Alternative. A full set of stage hydrographs at each index point for each modeled hydrology can be made available on CD by request.

Table E-10 Comparison of Group 1 Project Alternatives: Water Level Impacts for 1998 Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)			
		1998 Flood	Group 1 Alternatives		
			1-A	1-B	1-C
BF-1	Benson's Ferry	15.2	13.8	14.0	14.0
MR-2	Mokelumne River	10.9	8.8	9.2	9.2
SG-3	Snodgrass Slough	10.0	9.8	9.8	9.8
NH-4	New Hope	8.5	8.4	8.4	8.4
SF-5	SF[1] Mokelumne	7.5	7.4	7.4	7.4
SF-6	SF Mokelumne	7.3	7.3	7.3	7.3
SF-7	SF Mokelumne	7.3	7.2	7.2	7.2
NF-8	NF Mokelumne	8.2	8.2	8.1	8.2
NF-9	NF Mokelumne	7.4	7.3	7.3	7.3
NF-10	NF Mokelumne	7.2	7.2	7.2	7.2
MC-11	McConnell	47.3	47.3	47.3	47.3
TC-12	Twin Cities Road	28.3	28.3	28.3	28.3
LR-13	Lambert Road	10.9	10.9	10.9	10.9
PP-14	Point Pleasant	N/A	N/A	N/A	N/A
TT-15	Terminous Tract	7.2	7.2	7.2	7.2
NS-16	Confluence of NF and SF	7.1	7.1	7.1	7.1

[1] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

Table E-11 Comparison of Group 1 Project Alternatives: Water Level Impacts for 1999 Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)			
		1999 Flood	Group 1 Alternatives		
			1-A	1-B	1-C
BF-1	Benson's Ferry	14.2	13.0	13.2	13.2
MR-2	Mokelumne River	9.4	6.9	8.0	8.0
SG-3	Snodgrass Slough	7.0	6.9	6.9	6.9
NH-4	New Hope	5.9	5.8	5.9	5.9
SF-5	SF[1] Mokelumne	4.7	4.6	4.7	4.7
SF-6	SF Mokelumne	4.5	4.5	4.5	4.5
SF-7	SF Mokelumne	4.6	4.6	4.6	4.6
NF-8	NF Mokelumne	5.6	5.6	5.6	5.6
NF-9	NF Mokelumne	4.9	4.8	4.9	4.9
NF-10	NF Mokelumne	4.8	4.7	4.8	4.8
MC-11	McConnell	43.1	43.1	43.1	43.1
TC-12	Twin Cities Road	25.8	25.8	25.8	25.8
LR-13	Lambert Road	7.4	7.4	7.4	7.4
PP-14	Point Pleasant	N/A	N/A	N/A	N/A
TT-15	Terminous Tract	4.4	4.4	4.4	4.4
NS-16	Confluence of NF and SF	4.7	4.7	4.7	4.7

[1] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

Table E-12 Comparison of Group 1 Project Alternatives: Water Level Impacts for 2000 Flood Hydrology

Index Point	Location	Peak Stage (ft NGVD 29)			
		2000 Flood	Group 1 Alternatives		
			1-A	1-B	1-C
BF-1	Benson's Ferry	12.8	11.9	11.9	11.9
MR-2	Mokelumne River	8.9	7.1	8.0	7.9
SG-3	Snodgrass Slough	7.4	7.2	7.2	7.1
NH-4	New Hope	6.5	6.2	6.2	6.2
SF-5	SF[1] Mokelumne	5.9	5.7	5.8	5.8
SF-6	SF Mokelumne	5.7	5.6	5.7	5.7
SF-7	SF Mokelumne	5.6	5.6	5.6	5.6
NF-8	NF Mokelumne	6.2	6.0	6.1	6.0
NF-9	NF Mokelumne	5.8	5.6	5.8	5.7
NF-10	NF Mokelumne	5.5	5.6	5.6	5.6
MC-11	McConnell	41.9	41.9	41.9	41.9
TC-12	Twin Cities Road	24.8	24.8	24.8	24.8
LR-13	Lambert Road	7.9	7.9	7.9	7.9
PP-14	Point Pleasant	N/A	N/A	N/A	N/A
TT-15	Terminous Tract	5.6	5.6	5.6	5.6
NS-16	Confluence of NF and SF	5.5	5.5	5.5	5.5

[1] SF, NF: South Fork and North Fork of Mokelumne River, respectively.

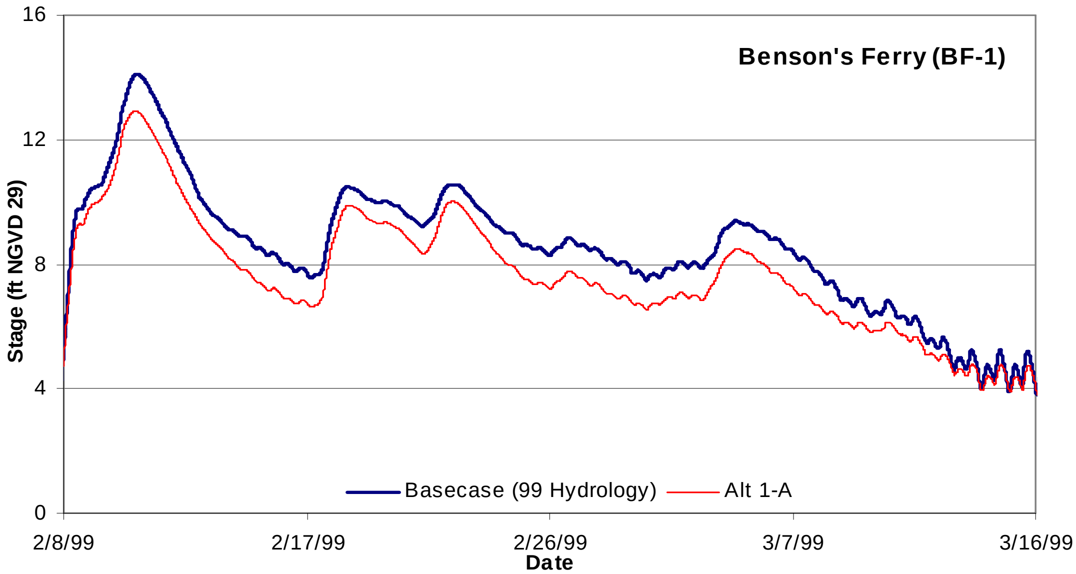


Figure E-33 Model results at Benson's Ferry for the 1999 flood hydrology showing the impact of Alternative 1-A compared to Alternative NP (No Project).

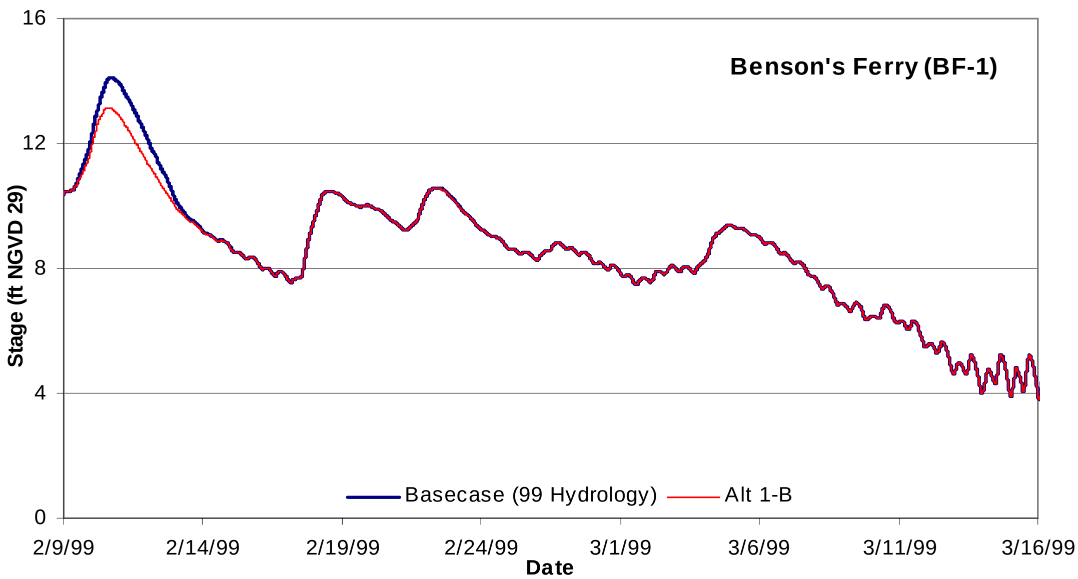


Figure E-34 Model results at Benson's Ferry for the 1999 flood hydrology showing the impact of Alternative 1-B compared to Alternative NP (No Project).

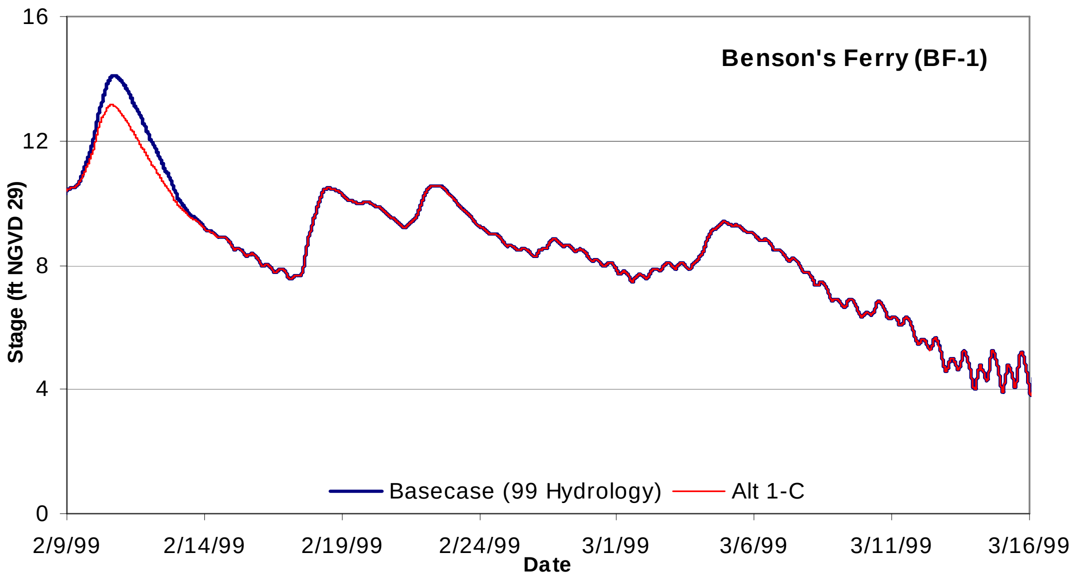


Figure E-35 Model results at Benson’s Ferry for the 1999 flood hydrology showing the impact of Alternative 1-C compared to Alternative NP (No Project).

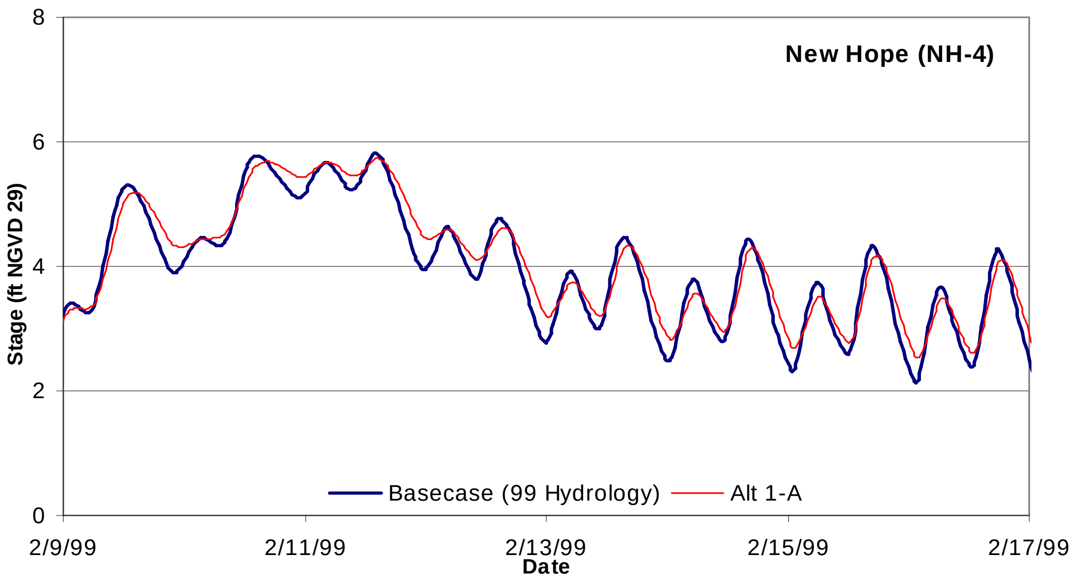


Figure E-36 Model results at New Hope for the 1999 flood hydrology showing the impact of Alternative 1-A compared to Alternative NP (No Project).

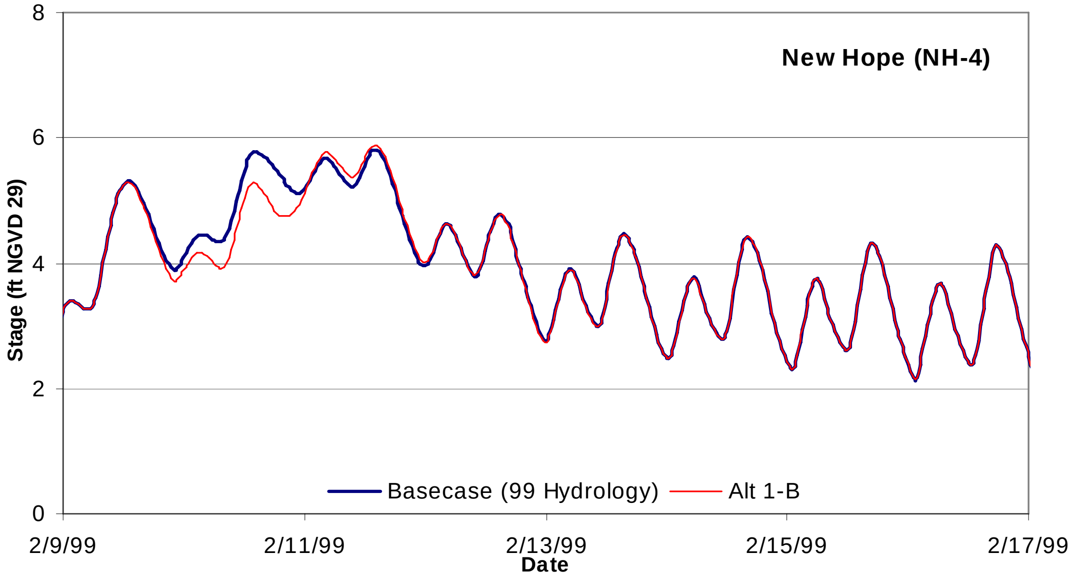


Figure E-37 Model results at New Hope for the 1999 flood hydrology showing the impact of Alternative 1-B compared to Alternative NP (No Project).

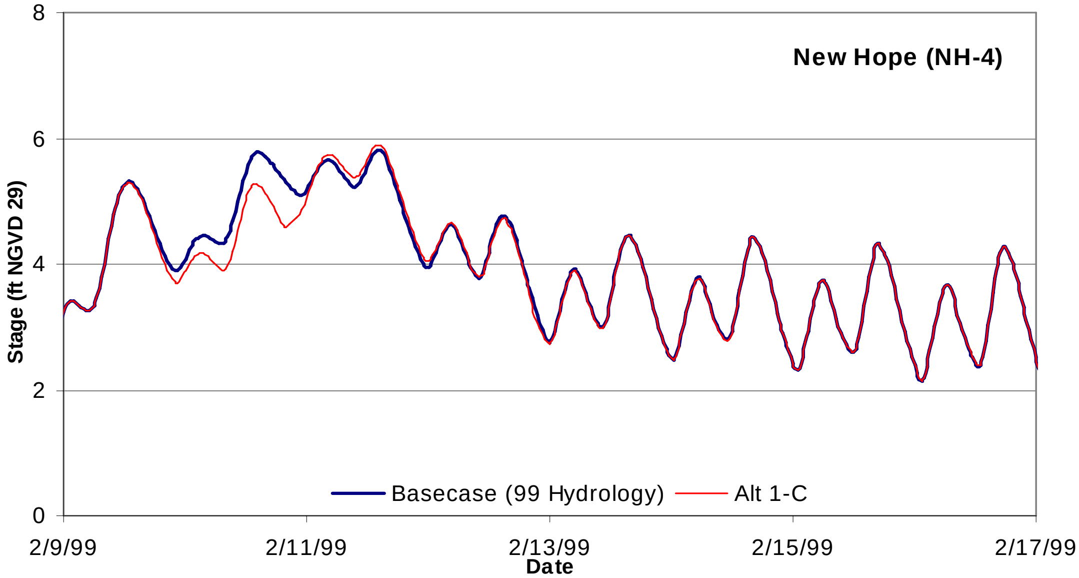


Figure E-38 Model results at New Hope for the 1999 flood hydrology showing the impact of Alternative 1-C compared to Alternative NP (No Project).

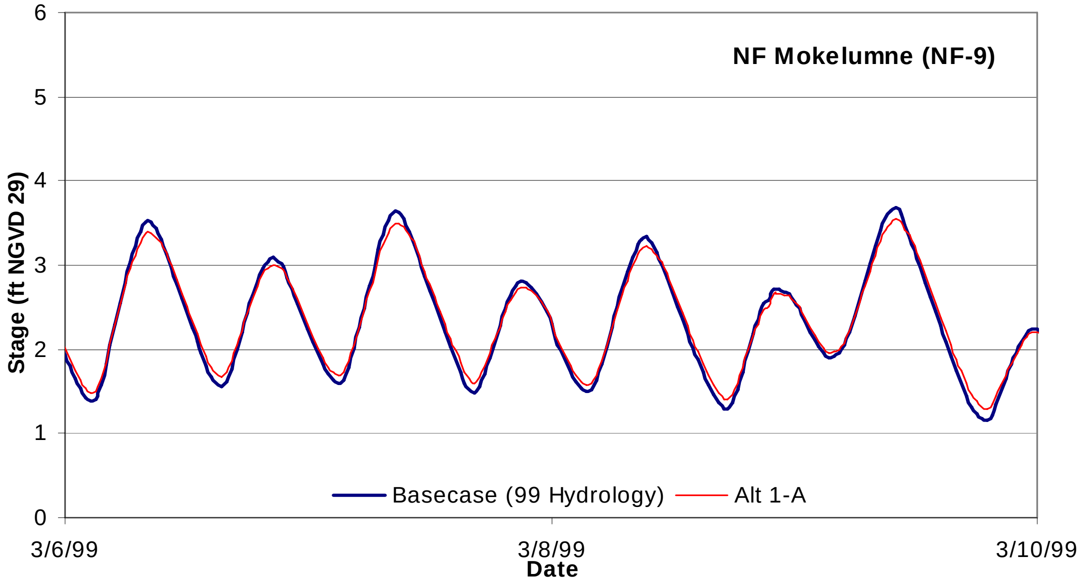


Figure E-39 Model results at NF-9 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-A compared to Alternative NP (No Project).

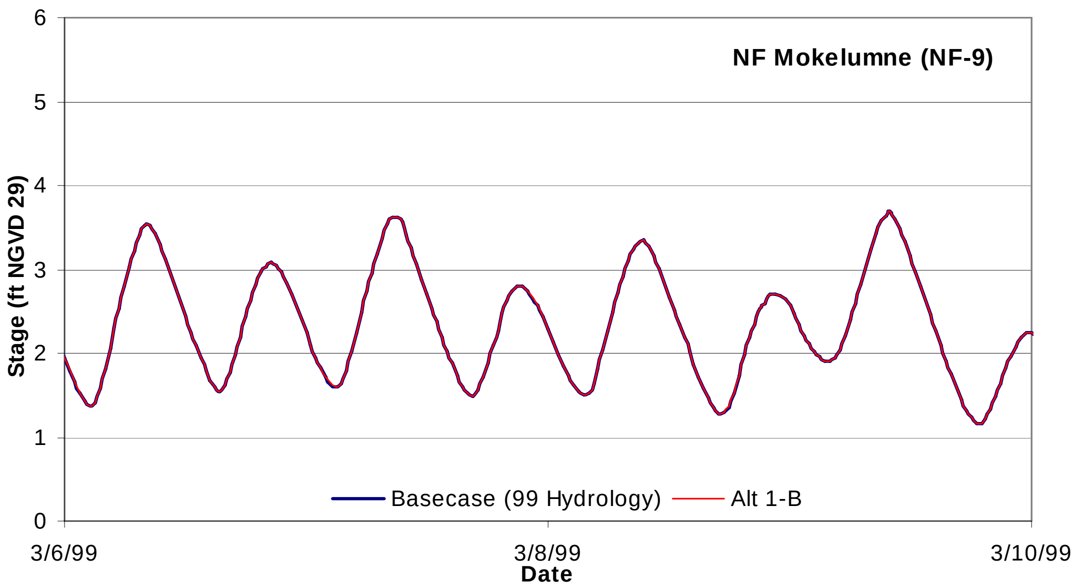


Figure E-40 Model results at NF-9 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-B compared to Alternative NP (No Project).

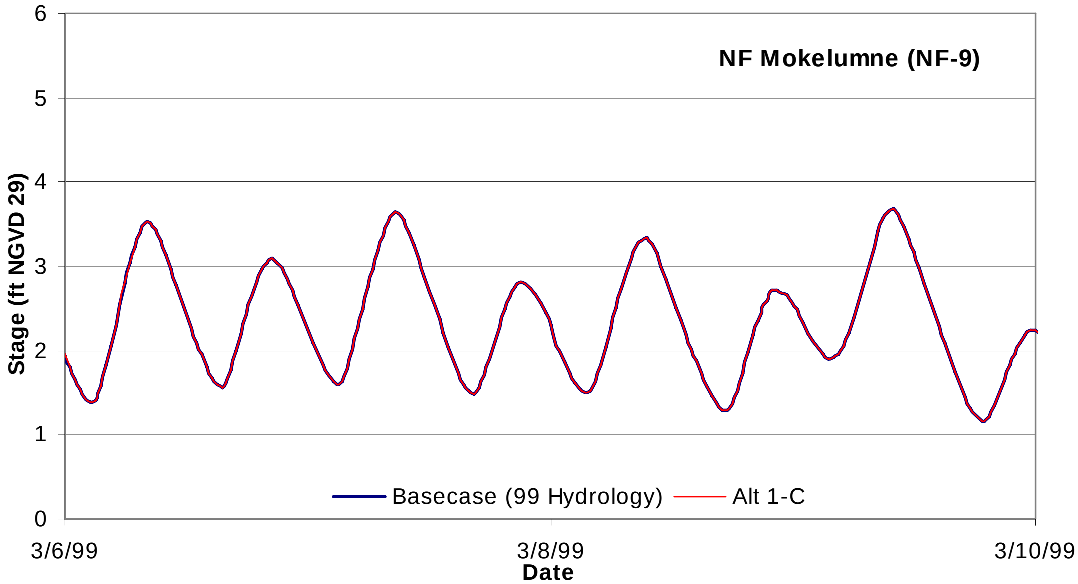


Figure E-41 Model results at NF-9 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-C compared to Alternative NP (No Project).

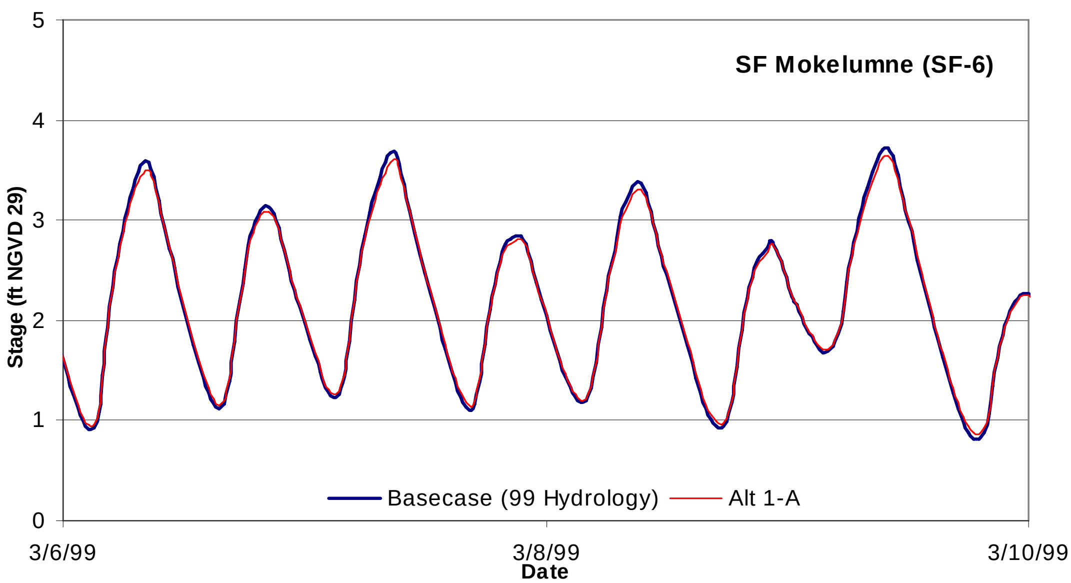


Figure E-42 Model results at SF-6 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-A compared to Alternative NP (No Project).

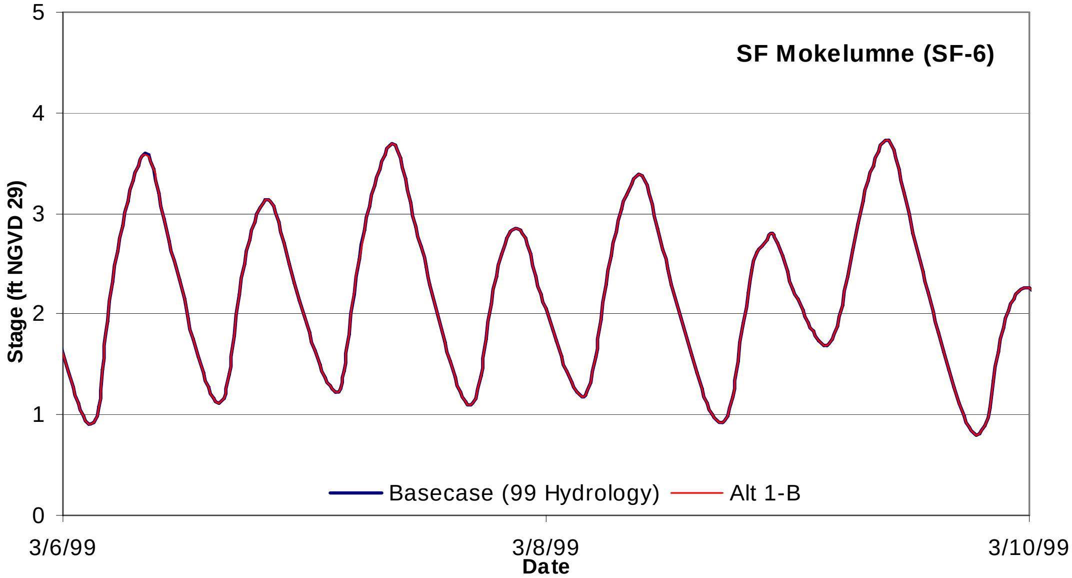


Figure E-43 Model results at SF-6 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-B compared to Alternative NP (No Project).

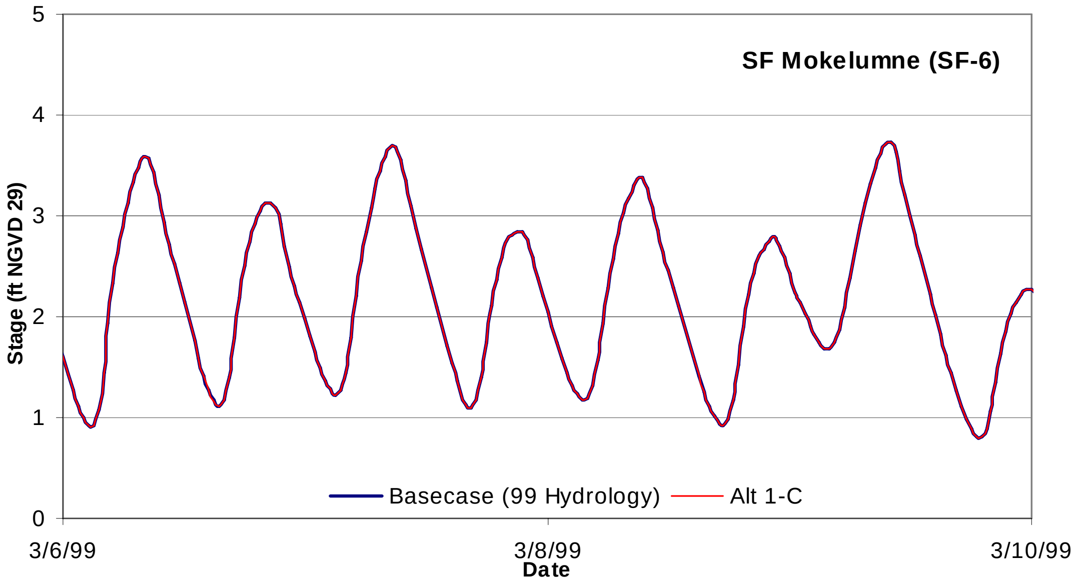


Figure E-44 Model results at SF-6 (for location, see Figure A-5) for the 1999 flood hydrology showing the impact of Alternative 1-C compared to Alternative NP (No Project).

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